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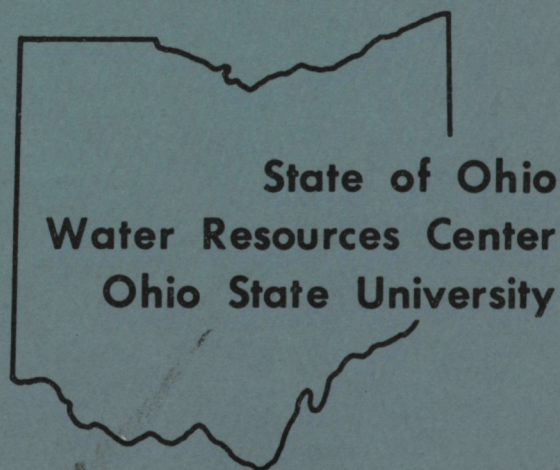
DEVELOPMENT OF COMPUTER PROGRAM FOR  
LINEARIZED SUBHYDROGRAPHS METHOD  
FOR URBAN RUNOFF DETERMINATION

By

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Project Completion Report

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#### DISCLAIMER

The opinions, findings and conclusions expressed in this report are those of the author and not necessarily those of the Office of Water Resources Research, U.S. Department of the Interior or the State of Ohio Water Resources Center.



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## ABSTRACT

A model for the simulation of urban runoff based on Linearized Subhydrographs Method of Urban Runoff Determination is developed for the planning and analysis of converging stormwater drainage systems in small and large urban areas. The Model simulates hydrographs for both continuously recorded rainstorm events and synthetic design hyetographs.

The model is applied to various urban watersheds with differing land use characteristics utilizing recorded hyetographs. The simulated hydrographs are then compared to recorded hydrographs.

The results indicate that when single event hyetographs are utilized, the simulated hydrographs for both the inlet and pipe conditions compare well to those results obtained from measured values.

The model structure and input requirements needed for the utilization of the program are presented. The limitations of the model are examined, and future research to improve this method is included.

KEY WORDS: Hydrologic Models, Urban Runoff, Hydrology, Drainage Systems, Stormwater Management, Hydrograph Simulation, Hydraulics



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## INTRODUCTION

The increasing level of urbanization in formerly nonurban areas has resulted in the need for better design and construction of adequate drainage systems. Over the years, various methods have been developed in the analysis and design of drainage systems. These procedures have evolved as a result of continual demand for the accurate determination of runoff and the need for the optimal design of sewer systems. However, most of these methods have been empirical in nature and although extensively used by engineers, their accuracy, from time to time, has been questioned. In addition, improper interpretation of the various empirical relations has led to the uneconomical design of drainage systems.

In recent years, modern hydrological and hydraulic concepts (31, 38, 41, 132)\* have been developed which utilize high speed computers and mathematical modeling techniques. These models are in general accurate and systematically eliminate uncertainties inherent in the data-gathering process and errors common to the design of drainage systems. Although these models simulate the runoff process to minute detail and determine pipe sizes accordingly, their use by designers has not been as widespread as anticipated mainly due to the complexity of the methods and difficulties experienced in gathering data for the calibration of the models. Oftentimes, because these methods are complex and sophisticated, they require excessive time to apply and thus become expensive when used as a design tool.

It is believed that Linearized Subhydrographs Method (19) would fulfill such needs as a practical tool in stormwater drainage planning.

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\* Numbers in parenthesis indicate references.

This method is a simplified procedure which generates a system hydrograph utilizing data readily available to the designers.

This study presents the development of a planning and analysis model, entitled SUBHYD, based on the Linearized Subhydrographs Method for determining runoff hydrographs in the analysis of drainage systems. The model is then tested by applying it to various watersheds with measured rainfall events and recorded runoff hydrographs. In addition, the results are compared to those obtained by simulation models that are currently in use.

The model structure and input requirements needed for the utilization of the program are presented in detail. The sensitivity of the various parameters (variables) incorporated in the model is established by comparing the simulated results to those obtained from watersheds with recorded hydrographs. The limitations of the model are examined, and recommended future research to improve this method is included.

## REVIEW OF RUNOFF METHODOLOGY

Over the years, empirical relationships have been developed to assist the planners and designers in determining peak runoff rates for a drainage basin (11,24,52,62,121). Many of these methods simplify the peak runoff calculations to yield a single value which then is used in the design of stormwater sewer systems. Perhaps the most widely used method in the design of stormwater sewers is the Rational or Lloyd Davies method developed by Kuichling (79). In applying the Rational Method to the design of storm sewers, the most difficult task is the accurate estimation of the rainfall intensity and the determination of proper runoff coefficients which are subject to individual judgment. The method entails the use of an average rainfall intensity for a duration equal to the time of concentration in the system. The time of concentration is defined as the time it takes the runoff water from the most remote part of the watershed to enter the inlet of the base. Various methods have been used for estimating time of concentration depending on the basin characteristics (4).

Modification of the Rational Method to form a runoff hydrograph was developed in the Rational "Rational" Method (121). This procedure assumes that by proportioning the basin area an inlet hydrograph could be developed based on the peak runoff rate. The time at which the peak runoff occurs is governed by the time of concentration for that basin. Although the peak runoff rate was determined by the Rational Method, this proportioning technique develops both the antecedent and receding limbs of the runoff hydrograph. The major advantage of this empirical technique is the generation of a runoff hydrograph which could then be combined to yield the system hydrographs needed in storm sewer design. The major disadvantage of the Rational

"Rational" Method is the laborious calculations needed in the development of runoff hydrographs.

The Unit Hydrograph Method as proposed by Eagleson (30) for storm sewer design is another empirical design procedure. This method determines the shape of the runoff hydrograph by dividing the ordinates of a storm hydrograph by the corresponding volume of rainfall excess. The unit hydrograph is equivalent to one inch of runoff from the area. It is pointed out that this procedure represents the unit hydrograph only for a given land use, rainfall excess duration and basin characteristics. Thus for the purpose of design, the unit hydrograph is magnified according to the storm pattern without changing its shape. Since the unit hydrographs change for each basin under consideration, the use of this method becomes cumbersome when applied to large watersheds.

Recent development of runoff analysis deals with the generation of mathematical models in the late 50's (132). These models were first initiated to analyze the local flooding of stormwater sewer overflows caused as a result of rapid urbanization. The models specifically were designed to evaluate the performance of existing systems and to determine the size of new systems.

A review of mathematical models used today indicates that these models fall under three general categories: planning models, design and analysis models, and real time simulation models.

The planning models are basically used to evaluate the impact of the change in land use and to show the effects of urbanization. The main objective of the planning models is aimed at minimizing the impact of urbanization by observing the changes in both water quality and quantity. These models also point out problem areas which might arise if land use patterns in the area are altered.



The second category contains models which perform the design and analysis aspect of stormwater sewer systems. These models may be used to accurately predict flood stage levels or determine design flows for a watershed. Design and analysis models are also used to check the adequacy of existing sewer systems and to determine the size of new stormwater systems to facilitate the increased volume of runoff due to urbanization.

In the third category, modeling the real time simulation of stormwater is presented by one of two methods. The first type of model monitors the existing sewer system and makes decisions as the sewer system is being hydraulically exceeded. With the help of on site retention basins, pumping stations and sewer by-passes, the diversion of flows through the stormwater system prevents problems which might otherwise occur. The second type of real time simulation involves the monitoring of rainfall in which the model then simulates the flow for determining whether or not certain storm sewered areas will need relief from the oncoming storm. These types of models are highly efficient and yield instantaneous results.

Although details of existing mathematical models can be found in literature, the following is a brief review of some significant models that have been developed for simulating runoff.

The U.S. Environmental Protection Agency's Stormwater Management Model (SWMM)(33, 34, 35, 38, 39, 40) is probably the most comprehensive model being used today. The model is well organized and describes the different components of the runoff phenomena very efficiently. It can simulate both the quantitative and qualitative aspects of runoff. Some of the subroutines (subprograms) that compose the SWMM Model are briefly described as follows:

Runoff Subroutine: This subprogram takes the hydrographic input and the rainfall hyetograph to develop the overland flow. Then by routing the runoff through gutters and pipes, the hydrographs and pollutographs are developed.

Transport Subroutine: The hydrographs and pollutographs previously developed are then routed through the main drainage system to an outlet discharge facility.

Receive Subroutine: If the outfall is a treatment plant, the subprogram simulates simple geometric storage and treatment facilities for their design. If the outlet is connected to a larger body of water, the model simulates the quality and quantity aspects over long periods of time.

One disadvantage of the model is the need for large amounts of data and the need to calibrate it against recorded results. Other disadvantages of this program are summed up in a quote from a recent publication by the U.S. Environmental Protection Agency (8).

"Fairly complete documentation of the model was published by the EPA, including a summary report, user's manual, verification and testing report and program listing. Unfortunately, no one of these reports presents a complete description of the theoretical bases and mathematical formulations of the model. The equations for some modeling phenomena are described in the user's manual, and some are not described at all. The reader must compare the two reports to obtain a fair understanding of the capabilities and limitations of the complete model and the meaning of the input data. Also, the user's manual includes much discussion of model verification and testing which adds to the report's bulkiness and makes it more difficult to find essential information for the preparation of the input data."

The Chicago Hydrograph Method (132,137) simulates the overland flow by taking into account infiltration rate, depression storage, gutter and pipe flow calculations. The model is specifically developed to design converging storm sewers on the basis of peak flows. It has the capability to simulate peak flows in an existing sewer network while also indicating surcharges of particular pipes in the network. One disadvantage of the model is its inability to calculate downstream control conditions, backwater effects, flow reversals and pressure flows. The Chicago Hydrograph Method was basically designed for single event calculations and has recently been modified to simulate both continuous storm patterns and quality calculations.

British Road Research Laboratory Model (RRL) (31,61) takes the basic hydrographical input and generates a design hydrograph which is used in the determination of pipe sizes. The parameters are used in a linear flow time-area technique similar to the one developed by Chow (21). Impervious and pervious areas are dealt with separately in the program for the generation of overland flow. The flow is then routed by steady state uniform flow to the inlet of the drainage area and a lag time is incorporated to set up the pipe hydrograph. It was determined that the RRL method can best simulate areas in which the amount of pervious area in a drainage basin equals the impervious area. Also, the design of sewer systems can only be simulated with rainstorms of a moderate intensity and duration. This hinders the design application in which the model was intended for.

The Colorado State University Urban Runoff Model (33) was developed to simulate the overflow conditions in sewer pipes and simulate nonsteady flow conditions. Although the model itself does not solve piping networks, the

concepts of a finite difference approach to the dynamic wave equations may in the future contribute to advancing urban runoff models. This model may be the forerunner for real time simulation of stormwater sewer systems.

The Corps of Engineers Storage, Treatment and Overflow Model (STORM) (33) is a planning tool used in determining storage requirements for wastewater treatment facilities. This model also simulates the quality aspect of stormwater and is used as a management tool. Continuous simulation data can be taken for both open channel and sewer pipe networks over long periods of time. The quality aspect of the runoff is simulated with the data obtained from land use characteristics.

The Minneapolis-St. Paul Urban Runoff Model (33) was developed to simulate real time conditions occurring in the main sewer systems. The program was set up on a small computer to separate excessive stormwater flowing into a sewage treatment plant. However, the use of the model was terminated because the program implementation was found to be too expensive. The idea of a real time simulation system for stormwater control may be economically sound in the future with the development of more efficient mini-computers.

The University of Cincinnati Urban Runoff Model (URCURM) (41, 110) is similar in structure to that of the Environmental Protection Agency's SWMM Model. The watershed is broken down into subcatchments whereby the program links each subcatchment with a storm sewer. The model is composed of three basic components such as the rainfall data, watershed characteristic and pipe data. The program uses Horton's equation for infiltration and depression storage and calculates overland flow. Each inlet hydrograph has an offset time where the average velocity in the pipe is found by a steady state uniform flow condition. The model is best suited for single event rainstorms

with continuous monitoring which determines differences between the measured runoff and simulated runoff. The quality aspects of this model are also very similar to that of the EPA's SWMM.

The University of Illinois Urban Storm Runoff Model, IUSR (142) is a highly sophisticated and accurate model which utilizes the St. Venant equations. Both overland and gutter flows are routed by a nonlinear kinematic wave approximation. The friction slope, for overland flow is calculated by the Darch-Weisbach equation, for gutter flow using Manning's equation. Sewer flows are routed by a nonlinear complete dynamic wave model which accounts for backwater effects. The model handles both subcritical and stable supercritical flows. At present the limitations of the model are that its accuracy has not been verified for steep slopes and that nonstable supercritical flows cannot be solved.

The Battelle Urban Wastewater Management Model (33) was developed to simulate flows from large urban watersheds with both quality and quantity being determined. The model simulates only the single event occurrences. This program is a versatile tool for design of a drainage system because sewer sizes, storage facilities, treatment plants and overflow facilities may be determined. The input data is simplified by classifying different urban characteristics upon which the calculations are based.

The Dorsch Consul Hydrograph - Volume Method (33) was developed in Europe and has been applied extensively there. The model simulates both converging and diverging sewer systems with both open channel and sewer pipe configurations using single event storms. The model uses the dynamic wave equation to simulate flow and can determine backwater effects.

As noted in the literature review, most of the models developed to date are elaborate and produce satisfactory results. However, the models as indicated require extensive data gathering and calibration schemes. Therefore, there is need to develop a computerized model which utilizes simplified input data and yields results which are comparable to those obtained by the more comprehensive and complex models. It is, therefore, proposed to develop a runoff model based on the Linearized Subhydrographs Method (19) which requires data that is already available to the designers.

The concept of the Linearized Subhydrographs Method (19) is based on the following principles: mass conservation of a subbasin, the linear variation of both the rising and receding limbs of the subhydrographs, and the superpositioning of these subhydrographs to form an inlet hydrograph. Considering the mass conservation concept, the volume of runoff being produced by a uniform rainfall remains essentially constant. This volume is then corrected by a runoff coefficient which takes into account all the "losses" occurring in the subbasin. The "losses" are composed of infiltration, depression storage and evaporation. Although it is known that the rising and receding limbs of the hydrograph are of a nonlinear nature, they are assumed to remain linear, since the subbasins are small the nonlinear or curvilinear variations can also be assumed to be small and, therefore, linear. The principle of superpositioning is utilized to combine the subhydrographs to form inlet hydrographs and to subsequently route the inlet hydrographs through the system.

The linearized subhydrograph is comprised of a time base which spans the time period from the beginning of rain to the time when the flow for that particular storm diminishes and becomes zero. The time base of the hydrograph is defined by the equation:

$$t_b = t_r + t_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad [1]^*$$

where

$t_b$  = time base of the hydrograph (minutes)

$t_r$  = duration of the storm (minutes)

$t_c$  = time of concentration for subbasin (minutes)

---

\*Numbers in brackets refer to equations

Equation 1 is utilized in forming three cases of linearized subhydrographs. The development of the three cases depends on the duration of the storm and time of concentration for a particular subbasin. The three cases of the linearized subhydrographs are shown graphically in Figure 1.

In Case I, the storm duration equals the time of concentration for that subbasin. In this case, the peak runoff occurs when the total flow from the subbasin contributes to the inlet of the subbasin. The peak runoff rate is given by:

$$q_p = C i A \quad . . . . . [2]$$

where:

$q_p$  = peak runoff rate (ft /sec)

$C$  = runoff coefficient

$i$  = intensity of rainfall (inches/hour)

$A$  = area of the subbasin (acres)

when,  $t_r = t_c$ , the time base of the subhydrograph becomes

$$t_b = t_r + t_c \quad . . . . . [3]$$

$$t_b = 2 t_r \quad . . . . . [4]$$

The total volume of runoff is then calculated by:

$$V = C i t_r A \quad . . . . . [5]$$

In Case II, the duration of the rainfall intensity is longer than the time of concentration for the subbasin. Therefore, the peak runoff rate is reached before the end of the storm. Thus, for a given time interval, the entire subbasin is contributing to the runoff. The duration of peak runoff



is equal to the quantity  $(t_r - t_c)$  with the runoff receding to zero in a specified time period equal to  $t_c$ . The peak runoff rate is defined by:

$$q_p = C i A \quad \dots \dots \dots [3]$$

when  $t_r > t_c$  the time base is:

$$t_b = t_r + t_c \quad \dots \dots \dots [2]$$

The volume of runoff is:

$$V = C i t_r A \quad \dots \dots \dots [5]$$

In Case III, the time of concentration for the subbasin is greater than the rainfall duration ( $t_r < t_c$ ). In this case, the entire subbasin is not contributing to the flow at the inlet before the storm ends. Therefore, the peak flow is adjusted according to the formula:

$$q_p = C i \left( \frac{2t_r}{t_r + t_c} \right) A \quad \dots \dots \dots [6]$$

When  $t_r < t_c$  the base time is

$$t_b = t_r + t_c \quad \dots \dots \dots [2]$$

The volume of runoff being:

$$V = C i t_r A \quad \dots \dots \dots [5]$$

It is noted that the total volume of runoff for a given storm event is assumed to be the same over the entire subbasin. Therefore, for a particular subbasin, this total volume of runoff is consistent with the mass conservation assumption.

In this method inlet subhydrographs are generated utilizing a single rainfall intensity which are then hydrologically routed. The

hydrologic routing is composed of hyetographical routing and topographical routing. With hyetographical routing, a chronological time lagging is assumed for each subhydrograph proportional to the duration of the rainfall increment. The topographical routing takes place after the development of the inlet hydrographs by lagging the inlet hydrographs according to the travel time. A schematic of hydrologic routing is shown in Figure 2. For the purpose of modeling, the three cases of the Linearized Subhydrographs Method are condensed into the following form:

Case A:  $t_r \leq t_c$

$$1) \text{ For } t \leq t_r, Y_t = i \left( \frac{2t_r}{t_r + t_c} \right) \frac{t}{t_r} \dots \dots \dots [7]$$

$$2) \text{ For } t > t_r, Y_t = i \left( \frac{2t_r}{t_r + t_c} \right) \left( \frac{t_r + t_c - t}{t_c} \right) \dots \dots \dots [8]$$

For Case B:  $t_r > t_c$

$$1) \text{ For } t \leq t_c, Y_t = i \left( \frac{t}{t_c} \right) \dots \dots \dots [9]$$

$$2) \text{ For } t_c < t < t_r, Y_t = i \dots \dots \dots [10]$$

$$3) \text{ For } t \geq t_r, Y_t = i \left( \frac{t_r + t_c - t}{t_c} \right) \dots \dots \dots [11]$$

The equation  $Q = CIA$  can be written in the following form to facilitate the modeling technique:

$$[Y_t][C_tA] = [Q_t] \text{ (cfs)} \dots \dots \dots [11a]$$

Expanding the above equation, the following set of equations are obtained:

$$Y_{01} C_1 A + 0 (C_2 A + C_3 A + \dots + C_t A) = Q_0$$

$$Y_{11} C_1 A + Y_{02} (C_2 A) + 0 (C_3 A + C_4 A + \dots + C_t A) = Q_1$$

$$Y_{21} C_1 A + Y_{12} (C_2 A) + Y_{03} C_3 A + 0 (C_4 A + C_5 A + \dots + C_t A) = Q_2$$

.

.

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.

$$0 (C_1 A + C_2 A + \dots + C_{t-1} A) + Y_{jt} C_t A = Q_j$$

where

$C_t$  = runoff coefficient at time  $t$

$A$  = area of the subbasin (acres)

$Q_t$  = discharge at time  $t$

$t$  = time after start of a storm (minutes)

$Y_t$  = modified intensity corresponding to the ordinate of the subhydrograph at time  $t$  (inches/hour).

$t_r$  = duration of the unit hyetograph (minutes)

$t_c$  = time of concentration (minutes)

$i$  = rainfall intensity (inches/hour).

The runoff coefficient used in the previous equations account for the "losses" between the rainfall and runoff in a particular subbasin. These "losses" differ from one subbasin to another with the geomorphological patterns of the subbasin. The abstractions take into consideration surface wetting, infiltration, depression storage and evaporation. It is noted that these "losses" become less apparent as the duration of the storm increases.

In the development of the Linearized Subhydrograph Method the runoff coefficient is an important factor in determining the rate of runoff. It is seen from the literature search that few studies relating the correlation of runoff coefficients to rainfall patterns for urban areas exist. However, in the development of the Linearized Subhydrograph Method, Hoad's runoff coefficients are adopted (44). In Figure 3, Hoad's test data gives the variation of  $C$  as a function of rainfall duration for both impervious and improved impervious conditions. The equations describing the relations are given as:

$$C = \frac{t}{t + 8} \text{ (impervious areas) } \dots \dots \dots [12]$$

$$C = \frac{0.5 t}{t + 15} \text{ (improved pervious areas) } \dots \dots \dots [13]$$

Applying these equations to a drainage basin, the weighted runoff coefficient can be computed by the following relationship:

$$C = x\left(\frac{t}{t + 8}\right) + (1 - x)\left(\frac{0.5t}{t + 15}\right) \dots \dots \dots [14]$$

where:

$C$  = Hoad's runoff coefficient

$t$  = duration of rainfall (minutes)

$x$  = fraction of impervious area for a particular subbasin

The time of concentration for a particular subbasin is generally defined as the time required for the surface runoff to flow from the most remote point of the subbasin to the outlet of the subbasin. In the Linearized Subhydrographs Method, the time of concentration may be termed as a time of equilibrium, where the rate of runoff is equal to the rate of rainfall supply for a uniform rainfall intensity. The time of concentration is, by using the kinematic wave theory, based on work given in Ref. (116). The equation used is as follows:

$$t_c = 0.93 \frac{L^{0.6} N^{0.6}}{i^{0.4} S^{0.3}} \dots \dots \dots [15]$$

where:

$t_c$  = time of concentration (minutes)

$L$  = maximum overland length of travel to the inlet of the subbasin (feet)

$N$  = Manning's overland flow coefficient

$i$  = excess rainfall intensity (inches/hour)

$S$  = average overland slope (feet per foot)

Table I gives the ranges of Manning's "N" overland coefficients used in Eq. 15 for a specified area imperviousness (22). The time of concentration for watersheds with sewer systems is obtained by summing the overland flow time (or inlet time) and the time of travel in the sewer system.



## DEVELOPMENT OF THE MODEL

The runoff model (SUBHYD) developed using the Linearized Subhydrograph Method is classified as an analysis and planning model for converging conduit systems. Although the program itself is documented in detail, the procedure used in the allocation of memory for the program is complex and involved. Therefore, changes to the program other than those specified are not recommended since these changes may produce results which may be in error.

The model utilizes a converging iterative scheme for the generation of hydrographs for a given watershed. A generalized flowchart of the program showing the converging procedure is given in Figure 4. The model is divided into a main program and eight subroutines. The subroutines are entitled DATA, HYET, INITIAL, KINWAV, SUBHY, OUTPUT, ROUT and VEL. The listing and description of each subroutine are given in the Appendix.

The first subroutine is used for the allocation of memory. The memory requirement depends on the maximum time allowed for the outflow hydrographs, the maximum number of subbasins contained in the design watershed, the maximum number of rainfall increments, and the maximum diameter to be designed. The number of subbasins contained within the watershed and the number of rainfall increments comprising the storm pattern are obtained readily from the available data. The maximum pipe diameter is conservatively assigned because this value has no effect on the actual design diameters calculated in the program. However, this value is needed to define the upper limit in the computations. The maximum time allowed by the hydrograph calculations will depend upon the geomorphology of the subbasins and the total storm duration. In analyzing areas smaller than ten acres, the

total storm duration which has units of minutes is multiplied by two. If the subbasin area is greater than ten acres, the total storm duration is multiplied by three. This simple procedure allows adequate storage allocation for the hydrograph calculations. The value for storage allocation is obtained by the following formula:

$$S_s = \text{MAXTIM} + (4 \times M) + (3 \times (\text{NN} \times \text{MAXTIM})) + \\ (3 \times (M \times \text{NN}) + (35 \times \text{NN}) + (14 \times \text{MAXDIA}) \dots \dots \dots [16]$$

where

$S_s$  = storage allocation

MAXTIM = maximum time allowed for runoff hydrograph

M = number of rainfall increments contained within the total storm pattern

NN = number of subbasins in the watershed

MAXDIA = maximum diameter to be designed.

Upon determination of storage allocation,  $S_s$ , its value is entered on the card labeled MAN00120 which has the 12th position in the source deck.  $S_s$  is an integer and starts in column 24 and ends with a closed parenthesis sign.

The first three cards in the data deck define the title, input and output control options and the extent of computations required in the analysis. The remainder of the cards are used to code data related to hyetograph input, watershed characteristics, and conduit type. The following is the sequence that is used for coding the data. Input formats used in the model are given in Fig. 5.



1. FIRST CARD (Title)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
TITLE	11 - 70	Alphanumeric - variable describing the watershed area (this card labels the data deck and generates the title for the output).

2. SECOND CARD (Rainfall Data Control)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
M	7 - 10	Integer - the number of rainfall increments contained in the total storm.
DUR	14 - 20	Real - The duration of equal rainfall increments (if the rainfall increments are not of equal duration this variable is set to 0.0)
RATIO	24 - 30	Real - in the absence of actual rainfall data the value of RATIO is the decimal equivalent of the time to peak to the total storm duration, further explanation of RATIO is given at the end of this section (if actual rainfall data is used the value of RATIO is 0.0).
DELTA	34 - 40	Real - time increment used in the subhydrograph calculations, usually one minute.
NOPT	47 - 50	Integer - option control card, if NOPT is equal to zero, actual rainfall data will be entered as input. If NOPT is greater than zero, a synthetic design hyetograph is being entered as input.
MAXTIM	56 - 60	Integer - maximum time limit allowed for the hydrograph analysis (in minutes)

3. THIRD CARD (Conduit Data)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
AINTER	4 - 5	Integer - time interval, in minutes, controls hydrograph output (the value of AINTER must be a multiple of DELTA)

THIRD CARD (continued)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
NFACT	9 - 10	Integer - option control for the generation of a general system hydrograph or a detailed system hydrograph (the distinction between the two is discussed at the end of this section). For a general system hydrograph simulation NFACT is set equal to zero. For a detailed system a hydrograph simulation NFACT is set equal to one.
NCHECK	14 - 15	Integer - an option control in the printing of output titles, for a general system hydrograph this variable is set equal to zero. For a detailed system this variable is set equal to one.
NCHX	19 - 20	Integer - this variable is entered as zero if the runoff coefficient is held constant, if the runoff coefficient changes with storm duration this variable is set equal to one.
AROC	24 - 30	Real - this variable generates the extent to which Hoad's runoff coefficient is calculated; this variable in most cases will be set equal to 150.
NN	34 - 36	Integer - this variable defines the number of subbasins contained in the watershed. (The number of subbasins and the number of conduits (pipes) must be equal otherwise an error message will be generated).
MAXDIA	41 - 45	Integer - this variable defines the maximum pipe diameter allowed in the memory allocation, this variable must be assigned with extreme care and its value should be on the conservative side.

The model is designed to accept rainfall data either from measured rainfall records or from synthetically derived hyetographs. The synthetic hyetographs can be calculated from statistically developed rainfall intensity - duration - frequency curves or rainfall depth - duration, frequency curves given in References (20, 132).

4. NEXT M CARDS (Rainfall Data)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
DEPTH	13 - 15	Real - this variable defines the values obtained from the conversion of rainfall increments to corresponding depths for a specific duration. The magnitude of DEPTH is determined by the following formula:  $\text{DEPTH}(\text{inches}) = \text{intensity}(\text{inches/hour}) * \frac{1 \text{ hr.}}{60 \text{ min.}} * (\text{Duration of the rainfall increments (minutes)})$ <p style="text-align: right;">. . . . [23]</p>
DURAT	30 - 29	Real - This variable defines the duration of rainfall increment in minutes.

5. NEXT NN CARDS (Drainage Basin Data)

<u>Variable</u>	<u>Columns</u>	<u>Description</u>
NKIND	2 - 3	Integer - this variable is equal to one and indicates that the information on the card is specified for the basins.
SIZE	6 - 11	Real - defines the subbasin area (acres)
ALENGT	16 - 22	Real - overland length of the drainage basin, obtained by scaling off the farthest distance from the boundary of the subbasin to the inlet (feet).
VALUE	24 - 29	Real - average overland slope of the subbasin in per cent.
ROUGH	32 - 39	Real - Manning's overland flow coefficient, representative values are given in Table I.
PER	42 - 47	Real - this variable defines the ratio of impervious area to the total subbasin area, PER is obtained from land use maps or by computing the percentage of impervious area, I, from the following formula developed by Environmental Protection Agency (EPA).

$$I = 9.6(P)^{(0.573-0.0391 \log_{10}(P))} \dots\dots [24]$$

where

I = imperviousness (percent)

P = population density in the developed  
portion of the urbanized area (people/acre)

Note: If discrepancies arise between these two techniques, impervious areas determined by the planimeter should be utilized.

NO	50 - 51	Integer - subbasin identification number
NTYPE	59 - 60	Integer - output control, if hydrograph at the inlet is requested NTYPE is set equal to one, if inlet hydrograph is not needed NTYPE is left blank.
DSPT	61 - 64	Alphanumeric - name assigned to each subbasin

#### 6. NEXT M CARDS (Conduit Data)

<u>Variable</u>	<u>Column</u>	<u>Description</u>
NKIND	2 - 3	Integer - this variable is set equal to two for conduit data
SIZE	6 - 11	Real - specifies conduit section to be designed, If set equal to: <ul style="list-style-type: none"> <li>- 1.0 circular conduit is designed;</li> <li>0.0 semielliptical conduit is designed;</li> <li>1.0 rectangular conduit is designed.</li> </ul>
ALENGT	16 - 22	Real - length of conduit assigned to each subbasin (feet)
VALUE	24 - 29	Real - slope of conduit (feet per one hundred feet)
ROUGH	32 - 39	Real - minimum value for the Manning's roughness coefficient, suggested values are given in Table II.
PER	42 - 47	Real - maximum value for the Manning's roughness coefficient suggested values are given in Table II.

NO	50 - 51	Integer - conduit identification number.
NENDD	53 - 54	Integer - defines conduit branch geometry, must have same value as variable NO.
NCOLL	56 - 57	Integer - designates collector conduit for conduit designated as NENDD above, its value is always greater than the value of NENDD.
NTYPE	59 - 60	Integer - output control, if pipe hydrograph of the subbasin is requested NTYPE is set equal to one, if pipe hydrograph is not needed NTYPE is left blank.
DSPT	61 - 64	Alphanumeric - name assigned to each pipe
PERCE	73 - 76	Real - used when combined sewer systems are being designed. The variable PERCE represents the ratio of sewerage flow to stormwater flow. In order to determine the value of PERCE two computer runs are used. The first computer run determines the maximum discharge from rainfall (PERCE is set equal to 1.0) and the second computer run develops the combined sewerage output after the ratio to total flow to stormwater flow is inputted.

It is noted that in using Manning's roughness coefficient, the maximum value is set equal to those given in Table II. The minimum value of the coefficient is set equal to listed minimum or average values shown in Table II. The description to use minimum or average values is left to the individual designer.

The simulation of runoff from a given watershed can be accomplished in two ways: a. General System Analysis, b. Detailed System Analysis.

The general system analysis is undertaken when pipe (conduit) data is not readily available and a quick approximation of the runoff hydrograph is needed. The simulated hydrographs represent runoff reaching the outlet of the watershed.

The detailed system is undertaken when the pipe system is included in the analysis and where hydrograph simulation is needed at each inlet of a subbasin. Sample outputs obtained from a general system analysis and a detailed system analysis are given in Figures 6 to 10 inclusive.

a. General System Analysis

In this analysis, only watershed characteristics are needed. This information can be obtained from topography maps, land use maps or aerial photographs. The following outlines the procedure and the sequence of control cards needed in the generation of hydrographs for a general system analysis.

1. FIRST CARD (Title) - previously defined

2. SECOND CARD (Rainfall Data Control)

M = previously defined

DUR = previously defined

RATIO = previously defined

DELTA = 1.0

NOPT = 0

MAXTIM = previously defined

3. THIRD CARD (Conduit Data Control)

AINTER = previously defined

NFACT = 1

NCHECK = 0

NCHEx = 0

AROC = 150.0

NN = 1

MAXDIA = previously defined

4. NEXT M CARDS (Rainfall Data)

26

All variables previously defined.

5. NEXT CARD (Drainage Basin Data)

All variables previously defined. (Only one card is required)

b. Detailed System Analysis

The procedure that is followed in a detailed system analysis is very similar to that of a general system analysis. However, in this case, data related to the pipe system (existing or proposed) and subbasin characteristics are needed. The information related to the piping system and subbasin characteristics can be obtained from existing sewer maps, topographic maps, land use maps and aerial photographs.

The watershed is divided into subbasins and an inlet is assigned to each. The number is the same as the subbasin it drains. The inlets are then connected by a pipe system in a converging manner. Each pipe is assigned a number starting from the most remote point of the watershed sequentially increasing the pipe numbers as they proceed through the watershed. The only geometry restriction in a piping network is that a pipe discharging into a downstream subbasin pipe must have a smaller number, NO, than the downstream pipe. The numbering of a pipe network for a given drainage basin is accomplished by first showing the schematics of the pipe network on the drainage map and then assigning numbers of each pipe in an increasing order as discussed above. The three examples that show the pipe numbering scheme are given in Figures 11 to 13. The corresponding coding forms for the input are given in Tables III, IV, and V. These are also the watersheds used in the verification of the model. It is noted that the first column of numbers

include the number of the pipe and the second and third columns (NENDD and NCOLL) give the direction of flow. In other words, pipe number 1 is carrying water from area 1 to area 2.

The following section summarizes the control cards needed in the utilization of this model for a detailed system analysis.

1. FIRST CARD (Title) - previously defined
2. SECOND CARD (Rainfall Data Control)
  - M = previously defined
  - DUR = previously defined
  - RATIO = previously defined
  - DELTA = 1.0
  - NOPT = 0
  - MAXTIM = previously defined
3. THIRD CARD (Conduit (Pipe) Data Control)
  - AINTER = previously defined
  - NFACT = 1
  - NCHECK = 0
  - AROC = 150.0
  - NN = previously defined
  - MAXDIA = previously defined
4. NEXT M CARDS (Rainfall Data) - previously defined
5. NEXT NN CARDS (Drainage Basin Data) - previously defined
6. NEXT NN CARDS (Pipe (Conduit) Data) - previously defined



Upon the allocation of memory and input data, the program proceeds to initialize all time and flow memory locations to zero. Calculations are then initiated to find the proper runoff coefficients and time of concentrations for the drainage system. The program generates the subhydrographs for each rainfall increment for a given subbasin. The subhydrographs are combined, using the principal of superpositioning to develop inlet hydrographs for the subbasins. The pipe hydrographs are developed by topographic routing through the piping network. The topographical routing is based on uniform flow calculations determined by Manning's equation. The pipe sizes are determined using peak flows. The following gives a listing of equations used in the model.

$$Q = \frac{1.49}{N} R^{2/3} S^{1/2} A \dots \dots \dots [25]$$

$$V_u = \frac{1.49}{N} R^{2/3} S^{1/2} \dots \dots \dots [26]$$

$$d = (2.16 \frac{N}{\sqrt{S}} q_p)^{3/8} \text{ (circular pipe sections) } \dots \dots \dots [27]$$

$$d = (2.12 \frac{N}{\sqrt{S}} q_p)^{3/8} \text{ (semi-elliptical pipe section) } \dots \dots [28]$$

$$d = (0.53 \frac{N}{\sqrt{S}} q_p)^{3/8} \text{ (rectangular pipe sections) } \dots \dots [29]$$

where

$Q$  = discharge ( $\text{ft}^3/\text{sec}$ )

$V_u$  = velocity of flow ( $\text{ft}/\text{sec}$ )

$A$  = cross sectional area ( $\text{ft}^2$ )

$N$  = Manning's roughness coefficient (Table III)

$R$  = hydraulic radius ( $\text{ft}$ )

$S$  = pipe slope ( $\text{ft}/\text{ft}$ )

$q_p$  = peak flow ( $\text{ft}^3/\text{sec}$ )

$d$  = diameter or size for a particular conduit (ft)

The variation of Manning's  $N$  with depth of flow (4) is incorporated in the conduit calculations and are shown in Figures 14, 15 and 16 along with variables related to conduit geometry. An averaging scheme is utilized in determining travel time and velocity of flow at a given depth. Since the conduit (pipe) sizes are unknown, the routing and determination of pipe sizes are established simultaneously through an iterative procedure. Pipe sizes are first determined for a flow increment and then checked with the previous value, if the newly designed pipes, as compared to the previously designed pipes, are not equal the process is repeated until the difference between them is within a ten percent margin. This procedure as it converges on the proper pipe sizes for the system also produces more realistic time of concentration for each subbasin. The time of concentration for a subbasin is composed of overland flow time and pipe travel time. Thus:

$$t_c = t_o + t_s \quad . . . . . [30]$$

where:

$t_c$  = time of concentration

$t_o$  = time of overland flow (Kinematic Wave Theory)

$t_s$  = travel time in pipe

In summary, the computations are accomplished in a continuously changing process until the pipe sizes fall within the ten percent marginal limit. It is felt that further reduction of this limit would result in a substantial increase in the computer time. The detailed flow chart and the program listing are given in the Appendix.

## SENSITIVITY ANALYSIS

The sensitivity analysis of the model consisted of applying the model to watersheds with measured rainfall events and recorded runoff hydrographs. Data related to the physical characteristics of the watershed was determined and were input to the model along with rainfall data and resulted in the generation of simulated hydrograph for the system as shown in Table VI. Basically, the analysis was divided into two general categories:

- 1) controlled variations using the General System Analysis
- 2) controlled variations using the Detailed System Analysis

Each category was analyzed by setting a "control" data package composed of percent imperviousness, Manning's "N" overland coefficient, design hyetograph, area, overland length, and overland slope. The values of the "control" data were held constant and a "control" hydrograph was generated. Then the variables were varied by increasing or decreasing their values by a certain percentage increment. The resulting hydrographs were then compared to the "control" hydrographs.

After varying all of the parameters, certain patterns became apparent. It was noted when parameters such as imperviousness, land slope, overland length and Manning's "N" were varied, significant changes in the peak discharges were not generated. In contrast, when the magnitude of the areas or the magnitudes of the rainfall events (hyetographs) were varied, a substantial increase or decrease resulted in the peak discharges. In other words, if a ten percent fluctuation was permitted for either the area or the hyetograph values the resulting hydrograph was proportionately greater than the control hydrograph values.

Thus, it became apparent that both the magnitude of the area and hyetograph pattern had significant effect on the simulated hydrographs.

The advantage of such a detailed sensitivity analysis leads to the fact that if given parameters are felt to be inconsistent, the resulting peak discharges may be modified accordingly. Also, certain confidence limits may be specified in the analysis based on the accuracy of these parameters.

## PRESENTATION AND DISCUSSION OF RESULTS

The simulated hydrographs obtained from the sensitivity analysis using the General System package and the Detailed System package are presented in this section along with the recorded hydrograph and the corresponding measured hyetographs. In addition, results obtained by using other models are presented.

The results obtained using the General System Analysis gave indications with regard to the capability of simulating an entire drainage basin without subdividing it. The basin characteristics for the three areas that were used in the General System Analysis are given in Table VII.

The first area shown in Table VII is from Oakdale, Chicago. This is an urban drainage basin with an area of 12.9 acres and an average land slope of one percent. A generalized map of the Oakdale, Chicago area is shown in Figure 17. In comparing the simulated hydrographs to the recorded hydrographs, the time to peak and the peak discharges from the model yielded good agreement. These comparisons are given in Figures 18 and 19.

The second and third areas given in Table VII are from Seattle, Washington. These two test sites are very similar in size, being 27.5 and 24.0 acres respectively. Also, the percentage of imperviousness of the area is high with flat overland slopes. Generalized maps of the watersheds are shown in Figures 20 and 21. Results from the Seattle, Washington areas indicate that when a General System Analysis is applied utilizing small rainfall intensities, the simulated hydrographs were less responsive than those with high rainfall intensities. This is due to the fact that the small rainfall intensities occur over such a long period of time that they have a tendency to spread the subhydrographs resulting in lagged superpositioning of the system hydrograph. The simulated hydrographs for the Seattle, Washington

area with lower rainfall intensities are given in Figure 22 and the hydrographs generated with the higher rainfall intensities are shown in Figure 23. The results with the higher rainfall intensities indicate that the simulated subhydrographs are more representative of the actual runoff. It can also be seen from Figure 23 that the general System Analysis can simulate peak flow conditions well, but the difference between the recorded and simulated hydrograph peaks are relatively significant. This is due to the fact that when there are no sewer system networks included in the analysis, the runoff simulation is essentially an overland flow process and does not represent the field conditions such as the existence of open channels and sewer systems. It is noted that since the general system analysis is essentially an overland flow case and the effect of flow conditions in small areas are not taken into consideration, the simulated hydrographs have delayed lower peaks and do not respond well to the variations that take place during the simulation process.

The Detailed System Analysis procedure was first applied to the Malvern Urban Test Area in Burlington, Ontario. The total basin area was 56.5 acres and was subdivided into ten subbasins ranging in size from 2.14 to 9.47 acres. The characteristics of the subbasins and corresponding pipe data are given in Table VIII. The results shown in Figures 24 and 25 indicate that in simulating the peak runoff values, the model yielded good agreement. However, the overall simulation when compared to the recorded hydrographs and the Environmental Protection Agency's SWMM is not consistent. The main reason for this inconsistency is due primarily to the difficulties experienced in scaling off hyetograph values from the original graphs, especially with the smaller intensities. It is noted that when the hyetograph with higher intensities was considered, much better simulation was obtained. Figure 26 shows that the simulated hydrograph deviates from the recorded hydrograph with the time to peak skewed to the right. This lack of time synchronization is due

to the zero rainfall intensity readings which cannot be properly simulated by the model. The program tends to smooth the outfall hydrographs and lower the peak discharges while shifting these peaks to the right. The results indicate that the SUBHYD model simulates single event rainstorms much better than the continuously recorded storms with intermittent zero rainfall events.

The second application of the Detailed System Analysis was performed for a watershed in Toronto, Canada. The basin area totaled 383.3 acres and was subdivided into 33 subbasins ranging in size from 3.9 to 25.9 acres. Both the basin characteristics and pipe data are given in Tables IX and X and a land use map of the test area is shown in Figure 27. From the results of Toronto, comparisons of the recorded and simulated hydrographs reinforce the fact that hyetographs containing zero rainfall increments do not simulate runoff as accurately as the single event rainfall patterns. Results shown in Figure 28 through 35 with single event rainfall patterns indicate that the simulated peak discharges both with regard to magnitude and time to peak are in close agreement with the recorded values. Storm patterns with intermittent zero rainfall increments like those shown in Figures 36 through 39. simulated hydrographs which were less sensitive and had lower peak discharges. The reason for the lack of simulation in these results is due to the inability of the model to simulate the non-rain portion of the hyetograph and the lack of mechanics incorporated in the model which does not deal with the minute details of the various runoff processes.

The last watershed tested using Detailed System Analysis option of SUBHYD model was the Bloody Run Test Site in Cincinnati, Ohio. This was the largest basin tested (2381.3 acres) and had subbasins ranging in size from 6.1 to 250.2 acres. The characteristics of the subbasins and the pipe data are shown in Tables XI and XII. Figure 40 indicates that the model did not

produce good results when compared to the recorded hydrographs. However, simulated hydrographs by the SUBHYD model when compared to the SWMM results indicate good agreement. It was concluded from the results that the recorded hydrograph values for this specific application were in error as pointed out in a study performed by the Environmental Protection Agency (35) Results shown in Figure 41, 42 and 43 are from a different study performed on the same urban test area which also includes results obtained by the University of Cincinnati Runoff Model. The results shown in Figures 41, 42 and 43 verify that when a single event rainstorm, without any zero rainfall increments, yield good simulation for both the peak discharges and the time to peak. The results also indicate that large watersheds, if subdivided into small subbasins, produce much better simulation.



## CONCLUSIONS AND RECOMMENDATIONS

A runoff model entitled SUBHYD is developed based on Linearized Subhydrograph Method. The model is designed to be used on computers comparable to those of IBM System/360-370 and UNIVAC 1108.

The model simulates overland flow and generates system hydrograph for a given drainage basin and subsequently computes conduit sizes for the peak flows. The data required for using the model consist of the following:

- a. Topographic map of the drainage basin
- b. Existing or planned stormwater system
- c. Land use maps or aerial photographs
- d. Synthetic or actual hyetographs

The following conclusions are reached based on the test application of the model to various watersheds:

1. The Linearized Subhydrographs Runoff Model promises to be a practical tool in the planning and analysis of stormwater systems due to its simple input requirements.
2. It is demonstrated that the SUBHYD Model simulates stormwater hydrographs for urban areas for both continuous and discrete storms. However, better simulation is obtained when continuous rainfall patterns are applied to urban drainage areas.
3. The model simulates peak flows well for hyetograph inputs with both continuous and discrete storms.

The following recommendations for further work may aid in the improvement of this model and its adoption in the planning and analysis of drainage systems:

1. Studies should be continued to improve the efficiency of the model in simulating hydrographs for continuously recorded rainfall patterns. Future work should be directed in obtaining better simulation for discrete storm events with intermittent zero rainfall increments.

2. The SUBHYD Model should further be developed as a planning tool for stormwater management including storm sewer design, storm-water pumping, equalization basins and storage facilities.
3. The incorporation of SUBHYD Model with a qualitative model to simulate stormwater quality would enhance the capability of the model.

In summary the SUBHYD Model developed in this report is a promising tool in simulating runoff for the planning and analysis of stormwater systems. Further research is needed to develop more comprehensive guidelines for using SUBHYD Model as a planning tool and expand its capability such that it can be used in all facets of stormwater management studies.

## TABLES



TABLE I

Range of Manning's "N" For Specified Imperviousness

<u>Imperviousness (%)</u>	<u>Manning's "N"</u>
0.0 - 15.0	0.370 - 0.350
16.0 - 25.0	0.220 - 0.200
26.0 - 35.0	0.120 - 0.100
36.0 - 45.0	0.085 - 0.075
46.0 - 60.0	0.060 - 0.055
61.0 - 80.0	0.040 - 0.035
81.0 - 100.0	0.020 - 0.018

TABLE II

Manning's Roughness Coefficient "N" for Conduit Material

	<u>Conduit Type</u>	<u>Minimum</u>	<u>Normal</u>	<u>Maximum</u>
1.	Cement - Neat Surface	0.010	0.011	0.013
2.	Mortar	0.011	0.013	0.015
3.	Concrete - Culvert straight	0.010	0.011	0.013
4.	Concrete - Culvert with bends, connection and some debris	0.011	0.013	0.014
5.	Concrete - finished	0.011	0.012	0.014
6.	Concrete - sewer with manholes, inlets	0.013	0.015	0.017
7.	Concrete - unfinished, steel form	0.012	0.013	0.014
8.	Concrete - unfinished, smooth wood	0.012	0.014	0.016
9.	Concrete - unfinished, rough wood	0.015	0.017	0.016
10.	Vitrified sewer	0.011	0.014	0.017
11.	Brickwork - lined with cement mortar	0.012	0.015	0.017
12.	Sanitary sewers coated with sewage slimes, with bends & connections	0.012	0.013	0.016
13.	Paved invert, sewer, smooth bottom	0.016	0.019	0.020
14.	Rubble masonry, cemented	0.018	0.025	0.030
15.	Corrugated metal - subdrain	0.017	0.019	0.021
16.	Corrugated metal - storm drain	0.021	0.024	0.030

Table III

## Pipe Numbering Code \*

<u>Pipe No.</u>	<u>NENDD</u>	<u>NCOLL</u>
1	1	2
2	2	3
3	3	5
4	4	5
5	5	6
6	6	10
7	7	8
8	8	9
10	10	10

Table IV

## Pipe Numbering Code\*\*

<u>Pipe No.</u>	<u>NENDD</u>	<u>NCOLL</u>
1	1	2
2	2	3
3	3	13
4	4	5
5	5	13
6	6	7
7	7	8
8	8	10
9	9	10
10	10	11
11	11	13
12	12	13
13	13	15
14	14	15
15	15	17
16	16	17
17	17	18
18	18	20
19	19	20
20	20	27
21	21	22
22	22	25
23	23	25
24	24	25
25	25	26
26	26	27
27	27	29
28	28	29
29	29	33
30	30	31
31	31	33
32	32	33
33	33	33

\*Refers to numbering code used for the Malvern Urban Test Area, Burlington, Ontario

\*\*Refers to numbering code used for Test Area, Toronto, Canada

Table V  
Pipe Numbering Code\*

Pipe No.	NENDD	NCOLL
1	1	3
2	2	3
3	3	4
4	4	5
5	5	6
6	6	9
7	7	9
8	8	9
9	9	10
10	10	11
11	11	12
12	12	13
13	13	13
14	14	15
15	15	18
16	16	17
17	17	18
18	18	19
19	19	28
20	20	21
21	21	22
22	22	27
23	23	27
24	24	27
25	25	27
26	26	27
27	27	28
28	28	30
29	29	30
30	30	31
31	31	33
32	32	33
33	33	36
34	34	36
35	35	36
36	36	38
37	37	38
38	38	38

\*Refers to numbering code used for the Bloody Run Urban Test Site, Cincinnati, Ohio.

TABLE VI. RESULTS OF SENSITIVITY ANALYSIS

				<u>Percent Change in Peak Discharge</u>	
				<u>38 Subbasins</u>	<u>1 Subbasin</u>
A)	<u>Imperviousness</u>				
1.	20% below the control	imperviousness		4.6 ↓	2.6 ↓
2.	10% below the control	imperviousness		2.3 ↓	1.6 ↓
3.	5% below the control	imperviousness		1.1 ↓	0.7 ↓
4.	5% above the control	imperviousness		1.1 ↑	2.9 ↑
5.	10% above the control	imperviousness		2.2 ↑	3.7 ↑
6.	20% above the control	imperviousness		4.7 ↑	4.2 ↑
B)	<u>Manning's "N" Overland Coefficient</u>				
1.	20% below the control	Manning's "N"		6.4 ↓	9.2 ↓
2.	10% below the control	Manning's "N"		3.1 ↓	3.1 ↓
3.	5% below the control	Manning's "N"		1.5 ↓	1.5 ↓
4.	5% above the control	Manning's "N"		1.4 ↑	1.5 ↑
5.	10% above the control	Manning's "N"		3.2 ↑	2.5 ↑
6.	20% above the control	Manning's "N"		6.4 ↑	5.4 ↑
C)	<u>Design Hyetograph</u>				
1.	20% below the control	hyetograph		23.3 ↓	22.5 ↓
2.	10% below the control	hyetograph		11.5 ↓	13.1 ↓
3.	5% below the control	hyetograph		6.6 ↓	8.4 ↓
4.	5% above the control	hyetograph		5.8 ↑	4.1 ↑
5.	10% above the control	hyetograph		11.5 ↑	9.4 ↑
6.	20% above the control	hyetograph		22.8 ↑	21.1 ↑
D)	<u>Area</u>				
1.	20% below the control	area		20.2 ↓	20.1 ↓
2.	10% below the control	area		10.1 ↓	10.1 ↓
3.	5% below the control	area		5.0 ↓	5.4 ↓
4.	5% above the control	area		5.0 ↑	4.7 ↑
5.	10% above the control	area		10.0 ↑	10.1 ↑
6.	20% above the control	area		20.0 ↑	20.2 ↑
E)	<u>Overland Length</u>				
1.	20% below the control	overland length		4.6 ↓	4.2 ↓
2.	10% below the control	overland length		2.3 ↓	3.4 ↓
3.	5% below the control	overland length		1.2 ↓	2.9 ↓
4.	5% above the control	overland length		1.1 ↑	0.7 ↑
5.	20% above the control	overland length		2.2 ↑	1.6 ↑
6.	20% above the control	overland length		4.3 ↑	2.6 ↑
F)	<u>Land Slope</u>				
1.	20% below the control	land slope		2.6 ↓	1.6 ↓
2.	10% below the control	land slope		1.2 ↓	0.7 ↓
3.	5% below the control	land slope		0.6 ↓	0.5 ↓
4.	5% above the control	land slope		0.5 ↑	2.3 ↑
5.	10% above the control	land slope		1.0 ↑	2.9 ↑
6.	20% above the control	land slope		1.9 ↑	3.1 ↑



Table VII

## Watershed Characteristics - General System Analysis

Location	Area (acres)	Overland Length (feet)	Slope (ft/100 ft)	Manning's "N"	Imperviousness %
1. Oakdale, Chicago	12.9	50.0	1.0	0.350	40.0
2. South Seattle, Washington	27.5	2400.0	0.5	0.020	90.0
3. South Center Tukwila, Washington	24.0	2700.0	0.5	0.020	97.0

Table VIII

## Characteristics of Malvern Drainage Basin (Burlington, Ontario)

Watershed Data

Subbasin Number	Area (acres)	Overland Length (feet)	Slope (ft/100 ft)	Manning's "N"	Imperviousness %
1	5.64	800.0	0.9	0.100	34.0
2	6.23	850.0	1.0	0.080	35.0
3	3.87	800.0	0.85	0.080	43.0
4	6.01	850.0	0.75	0.080	47.0
5	6.12	1000.0	1.2	0.100	31.0
6	2.26	350.0	0.7	0.100	36.0
7	9.47	1350.0	1.0	0.100	29.0
8	6.62	850.0	0.4	0.100	32.0
9	8.14	1100.0	0.9	0.100	23.0
10	2.14	600.0	0.7	0.060	51.0

Pipe Data

Pipe	Pipe Length (acres)	Slope (ft/100 ft)	Manning's N (Minimum)	Manning's N (Maximum)
1	720.0	1.0	0.012	0.016
2	160.0	1.3	0.012	0.016
3	420.0	1.1	0.012	0.016
4	550.0	0.75	0.012	0.016
5	440.0	1.2	0.012	0.016
6	560.0	0.7	0.012	0.016
7	870.0	1.0	0.012	0.016
8	220.0	0.2	0.012	0.016
9	320.0	1.1	0.012	0.016
10	200.0	0.9	0.012	0.016

Table IX  
Watershed Data - Toronto, Canada (104)

Subbasin Number	Area (acres)	Overland Length (feet)	Slope (ft/100 ft)	Manning's "N"	Imperviousness %
1	14.2	1150.0	0.9	0.055	49.0
2	7.8	1100.0	0.7	0.055	47.0
3	11.3	1600.0	1.9	0.055	49.0
4	8.7	850.0	0.9	0.055	47.0
5	9.7	1600.0	1.3	0.055	52.0
6	11.7	750.0	1.0	0.055	36.0
7	16.2	1200.0	1.5	0.055	42.0
8	5.6	800.0	0.8	0.055	47.0
9	13.6	1650.0	0.8	0.055	53.0
10	4.4	750.0	0.7	0.055	52.0
11	10.0	850.0	1.7	0.045	78.0
12	6.4	700.0	1.7	0.055	62.0
13	4.3	800.0	1.0	0.045	71.0
14	13.8	1200.0	1.4	0.055	51.0
15	15.3	1000.0	0.9	0.055	47.0
16	3.9	850.0	1.8	0.055	55.0
17	11.5	900.0	1.5	0.045	71.0
18	8.3	800.0	1.8	0.055	51.0
19	16.6	1400.0	1.0	0.055	50.0
20	18.6	1250.0	0.8	0.055	48.0
21	22.1	1300.0	1.4	0.055	47.0
22	22.1	1250.0	0.6	0.055	50.0
23	4.4	720.0	0.9	0.045	72.0
24	12.9	950.0	1.3	0.200	39.0
25	10.5	900.0	0.9	0.055	40.0
26	10.6	750.0	0.8	0.055	55.0
27	10.0	750.0	1.0	0.055	50.0
28	12.9	950.0	0.5	0.055	49.0
29	6.0	700.0	0.4	0.055	60.0
30	25.9	1800.0	1.1	0.055	41.0
31	7.7	850.0	0.7	0.200	6.0
32	5.4	650.0	0.9	0.055	44.0
33	20.9	1100.0	0.4	0.055	53.0

Table X

## Pipe System Data - Toronto, Canada (104)

Pipe Number	Pipe Length (feet)	Slope (ft/100 ft)	Manning's N (minimum)	Manning's N (maximum)
1	999.0	0.5	0.012	0.016
2	1592.0	0.64	0.012	0.016
3	900.0	0.43	0.012	0.016
4	1767.0	0.7	0.012	0.016
5	550.0	1.23	0.012	0.016
6	950.0	0.84	0.012	0.016
7	570.0	0.32	0.012	0.016
8	280.0	0.62	0.012	0.016
9	909.9	0.44	0.012	0.016
10	1120.0	2.04	0.012	0.016
11	398.0	1.12	0.012	0.016
12	300.0	3.07	0.012	0.016
13	982.0	0.79	0.012	0.016
14	239.0	0.33	0.012	0.016
15	700.0	0.26	0.012	0.016
16	237.0	0.41	0.012	0.016
17	451.0	0.37	0.012	0.016
18	212.0	0.49	0.012	0.016
19	541.0	1.27	0.012	0.016
20	600.0	0.80	0.012	0.016
21	830.0	0.97	0.012	0.016
22	1530.0	0.76	0.012	0.016
23	1920.0	0.55	0.012	0.016
24	640.0	0.50	0.012	0.016
25	690.0	1.02	0.012	0.016
26	570.0	0.55	0.012	0.016
27	280.0	0.53	0.012	0.016
28	810.0	0.30	0.012	0.016
29	650.0	0.60	0.012	0.016
30	200.0	0.41	0.012	0.016
31	860.0	0.40	0.012	0.016
32	350.0	0.35	0.012	0.016
33	700.0	0.50	0.012	0.016

Table XI

## Bloody Run Watershed Data - Cincinnati, Ohio (110)

Subbasin Number	Area (acres)	Overland Length (feet)	Slope (ft/100 ft)	Manning's "N"	Imperviousness %
1	176.0	5010.0	3.1	0.100	34.0
2	68.6	2625.0	3.1	0.100	29.0
3	73.2	2526.0	3.1	0.060	53.5
4	59.0	625.0	2.3	0.080	35.0
5	38.0	1062.0	11.0	0.080	37.0
6	33.4	1500.0	10.0	0.080	39.0
7	69.0	3062.5	2.2	0.045	73.0
8	250.2	4562.5	4.4	0.060	52.0
9	53.7	1375.0	11.3	0.060	57.0
10	45.2	750.0	2.7	0.060	59.0
11	38.4	1125.0	4.3	0.045	81.0
12	81.8	1375.0	3.2	0.060	55.0
13	49.8	1375.0	5.0	0.350	11.0
14	50.6	1125.0	5.4	0.100	30.0
15	48.0	1000.0	3.6	0.080	40.0
16	60.3	1125.0	4.3	0.060	49.0
17	21.9	750.0	5.9	0.060	56.0
18	29.9	1200.0	6.4	0.045	72.0
19	30.1	1562.5	8.3	0.080	45.0
20	88.3	1562.5	6.3	0.080	39.0
21	87.1	1062.5	2.1	0.045	61.0
22	39.3	1125.0	2.5	0.060	46.0
23	36.1	875.0	3.3	0.080	42.0
24	17.9	1125.0	4.0	0.200	25.0
25	42.5	1125.0	1.5	0.080	40.0
26	58.9	1250.0	2.2	0.080	40.0
27	6.1	400.0	2.0	0.080	40.0
28	91.2	2500.0	2.4	0.045	65.0
29	72.0	2687.5	3.1	0.200	21.0
30	116.0	1437.5	4.6	0.060	46.0
31	20.2	2312.5	3.5	0.100	35.0
32	54.0	2600.0	2.5	0.350	5.0
33	66.0	2312.5	4.2	0.350	5.0
34	62.5	3250.0	1.2	0.080	40.0
35	25.0	1300.0	2.7	0.200	17.0
36	122.0	710.0	7.3	0.060	50.0
37	50.1	2625.0	2.3	0.350	11.0
38	49.0	875.0	7.1	0.100	34.0

Table XII

## Bloody Run Pipe Data - Cincinnati, Ohio (110)

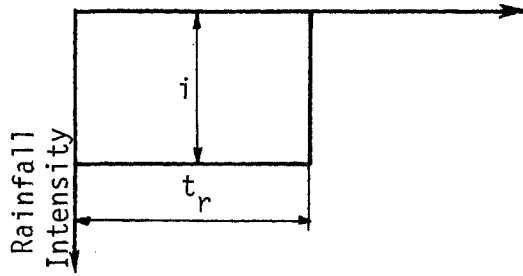
Pipe Number	Pipe Length (feet)	Slope (ft/100 ft)	Manning's N (minimum)	Manning's N (maximum)
1	2250.0	2.5	0.012	0.016
2	2400.0	3.0	0.012	0.016
3	450.0	3.0	0.012	0.016
4	1200.0	3.5	0.012	0.016
5	1450.0	4.0	0.012	0.016
6	1600.0	4.5	0.012	0.016
7	1250.0	3.5	0.012	0.016
8	1300.0	4.5	0.012	0.016
9	650.0	2.0	0.012	0.016
10	900.0	2.0	0.012	0.016
11	400.0	3.0	0.012	0.016
12	1200.0	3.0	0.012	0.016
13	400.0	3.5	0.012	0.016
14	1500.0	2.5	0.012	0.016
15	800.0	2.0	0.012	0.016
16	1600.0	2.0	0.012	0.016
17	600.0	2.0	0.012	0.016
18	1000.0	2.0	0.012	0.016
19	2600.0	3.5	0.012	0.016
20	1400.0	3.0	0.012	0.016
21	1200.0	2.0	0.012	0.016
22	850.0	2.0	0.012	0.016
23	900.0	2.0	0.012	0.016
24	600.0	2.5	0.012	0.016
25	150.0	2.0	0.012	0.016
26	200.0	2.5	0.012	0.016
27	400.0	2.0	0.012	0.016
28	1600.0	2.0	0.012	0.016
29	200.0	3.0	0.012	0.016
30	1400.0	3.0	0.012	0.016
31	2000.0	3.5	0.012	0.016
32	200.0	2.0	0.012	0.016
33	900.0	2.0	0.012	0.016
34	20.0	3.2	0.012	0.016
35	400.0	3.0	0.012	0.016
36	70.0	3.5	0.012	0.016
37	200.0	3.2	0.012	0.016
38	40.0	3.5	0.012	0.016



## FIGURES



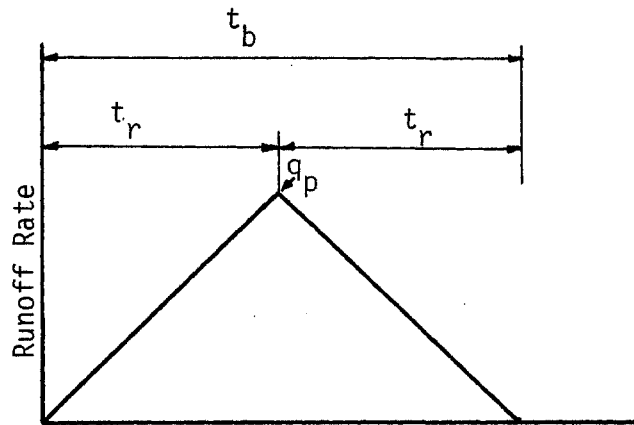




### Unit Hyetograph

$t_r$  = storm duration

$i$  = storm intensity

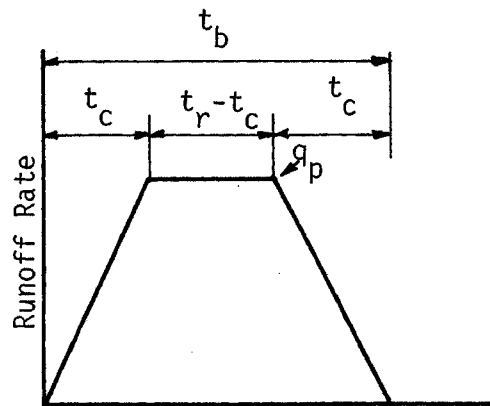


Case 1.  $t_r = t_c$

$$q_p = CiA$$

$$t_b = 2t_r$$

$$V = Cit_r A$$

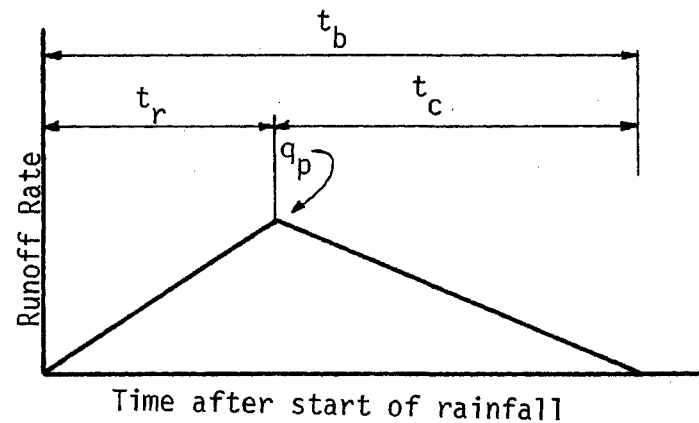


Case 2.  $t_r > t_c$

$$q_p = CiA$$

$$t_b = t_r + t_c$$

$$V = Cit_r A$$



Case 3.  $t_r < t_c$

$$q_p = CiA \frac{2t_r}{t_r + t_c}$$

$$t_b = t_r + t_c$$

$$V = Cit_r A$$

Figure 1 Schematics of Linearized Subhydrographs

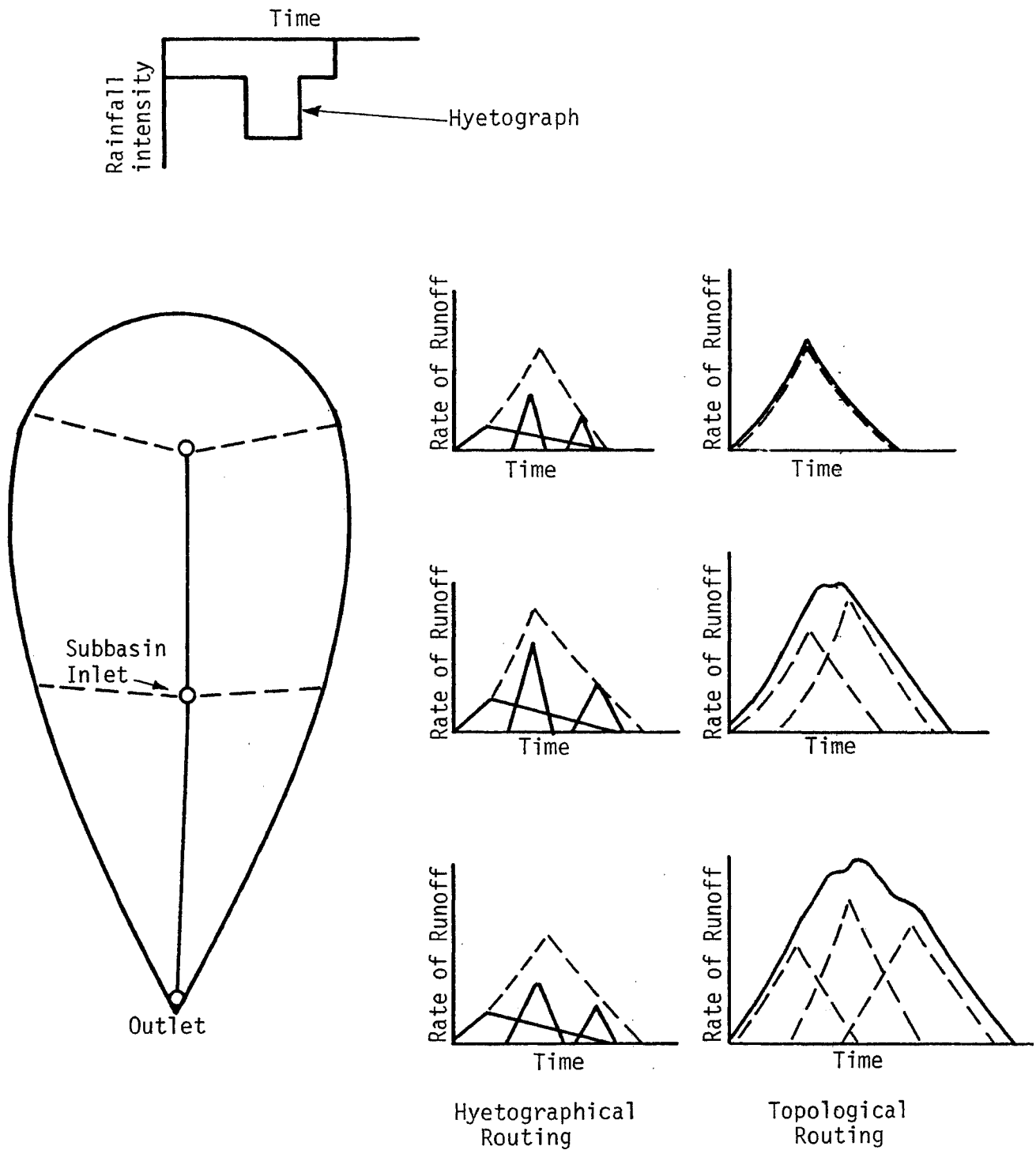


Figure 2 Schematics of the Hydrologic Routing Process

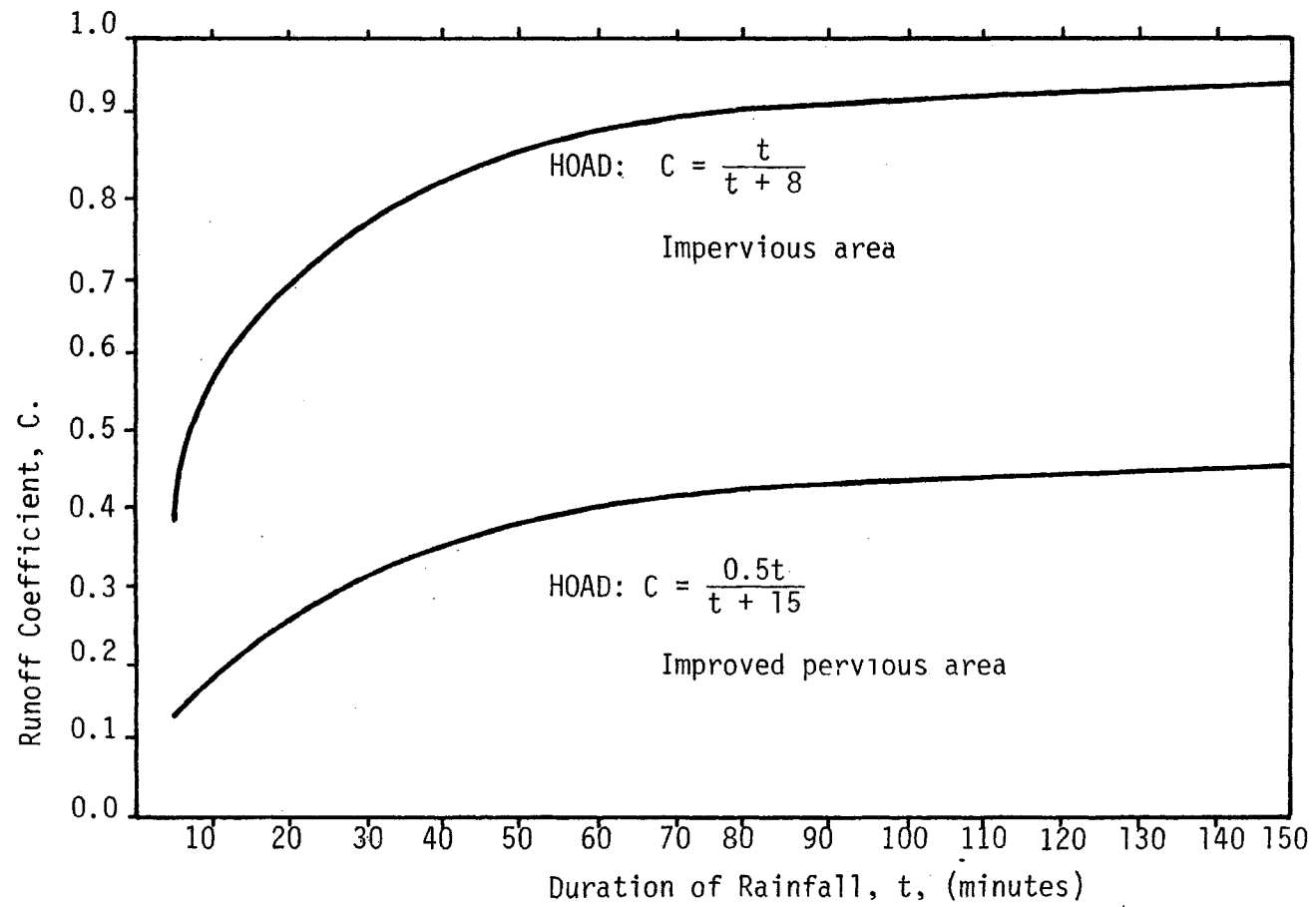


Figure 3 Runoff Coefficient versus Rainfall Duration After Hoad (44)

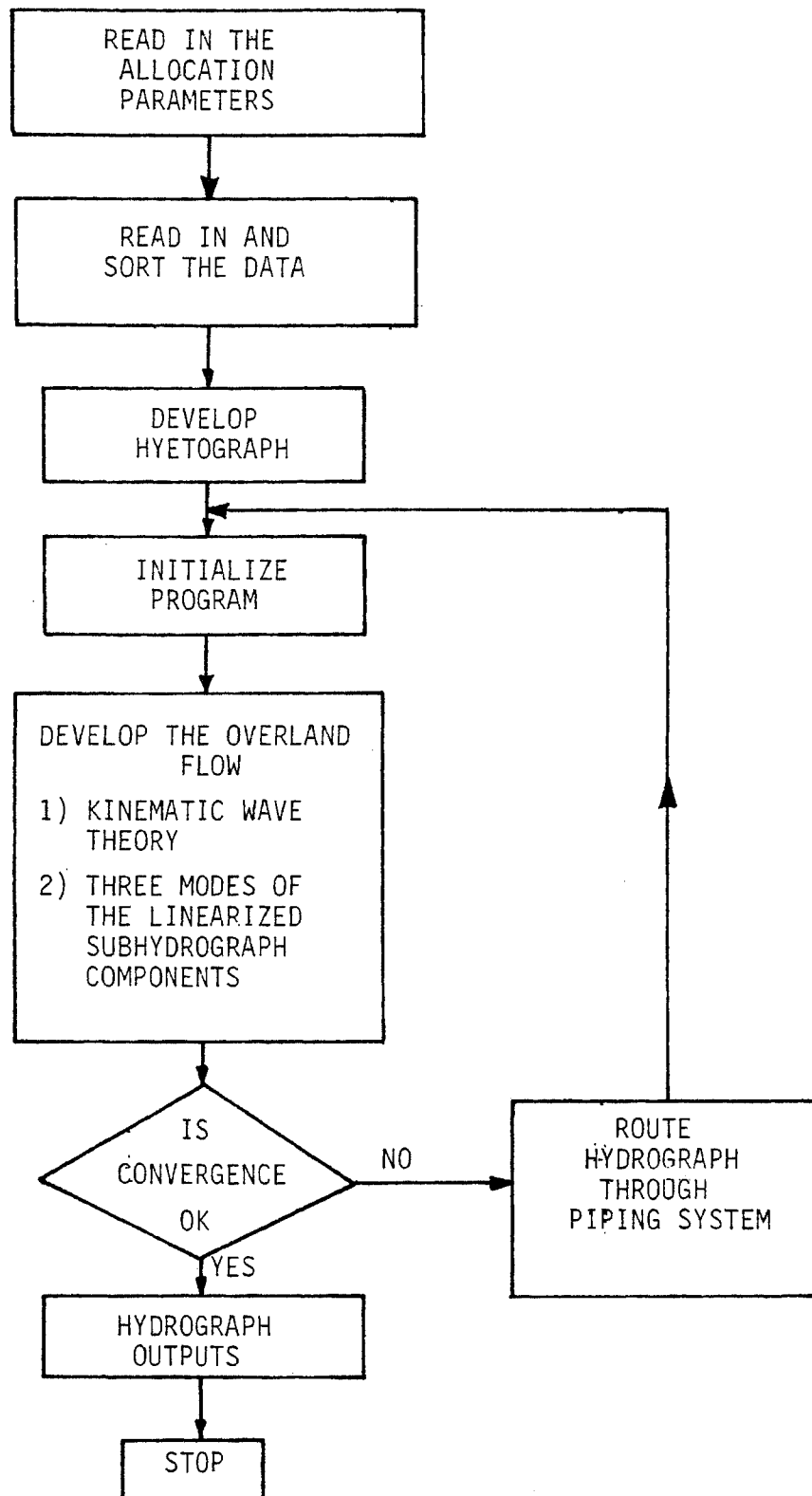


Figure 4 "SUBHYD" Model Generalized Flowchart

1234567891011121314151617181920212223242526272829303132333435363738394041424344454647484950515253545556575859606162636465666768697071727374757677787980																													
10X		TITLE (15A4)																											
6X	M14	3X	DUR F7.4		3X	RATIO F7.4		3X	DELTA F7.4		6X	NOPT I4	5X	MAXIM I5															
3X		3X		3X		3X	AROC F7.1		3X	NN I3	4X	MAXDIA I5																	
AINTER 12		NFACT 12		NCHECK 12		NCHEX 12																							
1X	2X	SIZE F6.2		4X	ALENGT F7.2		1X	VALUE F6.4		2X	ROUGH F8.4		2X	PER F6.2		2X	No I2	1X		1X	1X			DSPT A4	8X	PERCE F4.2			
KIND 12																	NENDD 12	NCOLL 12	NTYPE 12										
12X			DEPTH F8.4			10X			DURAT F10.4																				

Figure 5 Input Formats for the "SUBHYD" Model

## SYSTEM HYDROGRAPH, INLET NO. 1

DRAINAGE AREA = 12.9ACRES, MANNINGS N = 0.350, OVERLAND LENGTH = 50. FT.  
 SLOPE = 0.010FT/FT, FRACTION OF IMPERVIOUS SURFACE = 0.40

TIME (MINUTES)	SYSTEM DISCHARGE (CFS)
2.0	0.54
4.0	0.78
6.0	0.74
8.0	0.69
10.0	1.18
12.0	1.11
14.0	1.31
16.0	4.02
18.0	4.15
20.0	4.28
22.0	4.41
24.0	4.54
26.0	6.93
28.0	10.03
30.0	13.12
32.0	16.22
34.0	17.72
36.0	17.87
38.0	17.41
40.0	16.52
42.0	15.70
44.0	13.99
46.0	12.65
48.0	11.72
50.0	9.91
52.0	8.29
54.0	6.91
56.0	6.76
58.0	10.14
60.0	10.43
62.0	9.33
64.0	8.74
66.0	7.58
68.0	6.69
70.0	5.52
72.0	4.57
74.0	3.62
76.0	2.67
78.0	1.86
80.0	1.26
82.0	0.90
84.0	0.70
86.0	0.55
88.0	0.41
90.0	0.32
92.0	0.24
94.0	0.18
96.0	0.12
98.0	0.07
100.0	0.02
102.0	0.01

Figure 6 Example of Hydrograph Output - General System Analysis

## INLET HYDROGRAPH, INLET NO. 4

DRAINAGE AREA = 55.0ACRES, MANNINGS N = 0.080, OVERLAND LENGTH = 625. FT.  
 SLOPE = 0.023FT/FT, FRACTION OF IMPERVIOUS SURFACE = 0.35

TIME (MINUTES)	INLET DISCHARGE (CFS)
5.0	0.24
10.0	0.55
15.0	0.92
20.0	1.29
25.0	1.58
30.0	2.62
35.0	3.22
40.0	3.92
45.0	4.55
50.0	4.65
55.0	4.87
60.0	4.88
65.0	5.08
70.0	5.53
75.0	5.62
80.0	5.47
85.0	5.75
90.0	5.50
95.0	5.62
100.0	5.35
105.0	5.04
110.0	4.72
115.0	4.63
120.0	4.53
125.0	4.22
130.0	3.71
135.0	3.20
140.0	2.71
145.0	2.25
150.0	1.93
155.0	1.60
160.0	1.34
165.0	1.11
170.0	0.92
175.0	0.77
180.0	0.62
185.0	0.47
190.0	0.33
195.0	0.23
200.0	0.15
205.0	0.08
210.0	0.05
215.0	0.04
220.0	0.03
225.0	0.02
230.0	0.02
235.0	0.01

Figure 7 Example of Hydrograph Output - Subbasin Inlet

THIS IS A CIRCULAR PIPE WITH THE  
PIPE HYDROGRAPH, PIPE NO. 30

PIPE LENGTH= 1400.0 FT, PIPE DIA. = 33.35 INCHES, SLOPE = 3.0 PERCENT  
TRAVEL TIME IN PIPE IS 2.25 MINUTES

TIME (MINUTES)	PIPE DISCHARGE (CFS)
7.2	0.71
12.2	2.92
17.2	6.66
22.2	11.33
27.2	17.30
32.2	24.93
37.2	33.69
42.2	42.93
47.2	52.28
52.2	60.25
57.2	65.80
62.2	68.85
67.2	70.94
72.2	74.11
77.2	76.03
82.2	80.52
87.2	82.58
92.2	85.22
97.2	87.14
102.2	87.28
107.2	85.39
112.2	82.61
117.2	78.84
122.2	77.41
127.2	74.75
132.2	70.62
137.2	64.72
142.2	57.86
147.2	51.01
152.2	44.81
157.2	39.21
162.2	34.45
167.2	30.06
172.2	26.00
177.2	22.42
182.2	19.22
187.2	16.46
192.2	13.98
197.2	11.82
202.2	10.00
207.2	8.44
212.2	7.16
217.2	5.57
222.2	2.94
227.2	2.70
232.2	2.47
237.2	2.26
242.2	2.09
247.2	1.93
252.2	1.80

Figure 8 Example of Hydrograph Output - Circular Pipe



THIS IS A SEMI-ELLIPTICAL PIPE WITH  
 R1 = 25.23 INCHES R2 = 6.66 INCHES R3 = 21.04 IN  
 R4 = 6.66 INCHES R5 = 4.20 INCHES  
 PIPE HYDROGRAPH, PIPE NO. 19

PIPE LENGTH= 2600.0 FT, PIPE DIA. = 20.19 INCHES, SLOPE = 3.5 PERCENT  
 TRAVEL TIME IN PIPE IS 5.40 MINUTES

TIME (MINUTES)	PIPE DISCHARGE (CFS)
10.4	0.11
15.4	0.84
20.4	2.22
25.4	3.76
30.4	5.51
35.4	8.06
40.4	11.01
45.4	12.78
50.4	16.64
55.4	19.28
60.4	20.66
65.4	21.17
70.4	21.44
75.4	21.95
80.4	22.17
85.4	22.85
90.4	23.65
95.4	24.26
100.4	25.12
105.4	24.85
110.4	24.02
115.4	23.06
120.4	21.91
125.4	21.00
130.4	20.20
135.4	18.87
140.4	16.90
145.4	14.70
150.4	12.68
155.4	10.87
160.4	9.32
165.4	7.88
170.4	6.54
175.4	5.35
180.4	4.39
185.4	3.60
190.4	2.94
195.4	2.37
200.4	1.90
205.4	1.50
210.4	1.14
215.4	0.72
220.4	0.07
225.4	0.04
230.4	0.03
235.4	0.02
240.4	0.02
245.4	0.01
250.4	0.01
255.4	0.01

Figure 9 Example of Hydrograph Output - Semielliptical Pipe

THIS IS A RECTANGULAR TRUNK SEWER WITH THE  
 HEIGHT = 12.04 INCHES AND THE LENGTH = 24.08 INCHES  
 PIPE HYDROGRAPH, PIPE NO. 4

PIPE LENGTH= 1200.0 FT, SLOPE = 3.5 PERCENT  
 TRAVEL TIME IN PIPE IS 2.12 MINUTES

TIME (MINUTES)	PIPE DISCHARGE (CFS)
7.1	0.55
12.1	1.57
17.1	2.65
22.1	3.85
27.1	5.86
32.1	8.10
37.1	10.23
42.1	12.63
47.1	15.05
52.1	16.33
57.1	17.31
62.1	17.97
67.1	18.95
72.1	20.73
77.1	21.97
82.1	22.38
87.1	23.73
92.1	25.06
97.1	25.38
102.1	25.39
107.1	25.32
112.1	25.10
117.1	25.20
122.1	25.30
127.1	24.79
132.1	23.63
137.1	22.30
142.1	21.00
147.1	19.76
152.1	18.59
157.1	17.45
162.1	16.40
167.1	15.45
172.1	14.58
177.1	13.77
182.1	12.98
187.1	12.20
192.1	11.43
197.1	10.72
202.1	10.07
207.1	9.46
212.1	8.90
217.1	8.40
222.1	7.93
227.1	7.48
232.1	7.05
237.1	6.64
242.1	6.25

Figure 10 Example of Hydrograph Output - Rectangular Pipe

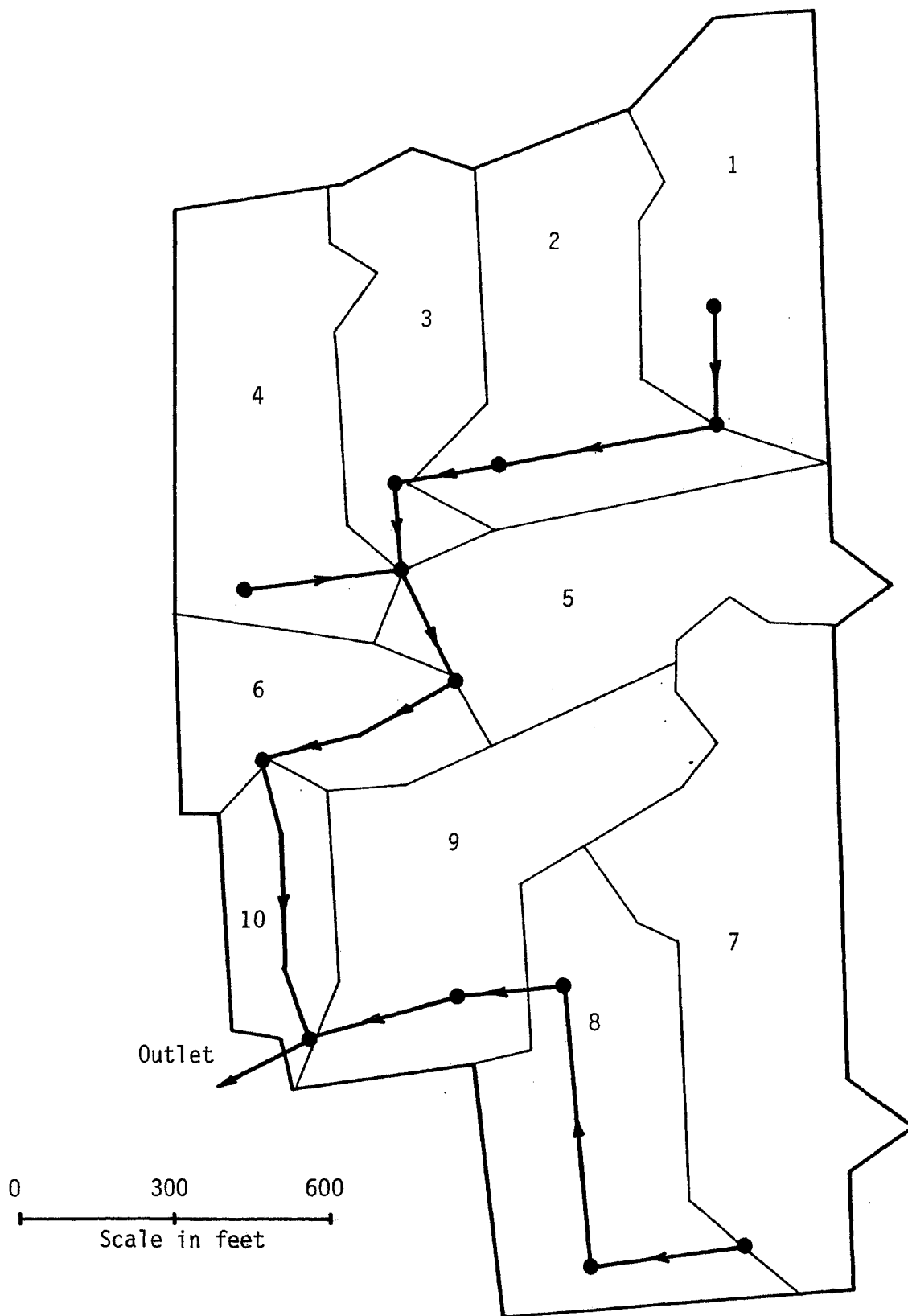
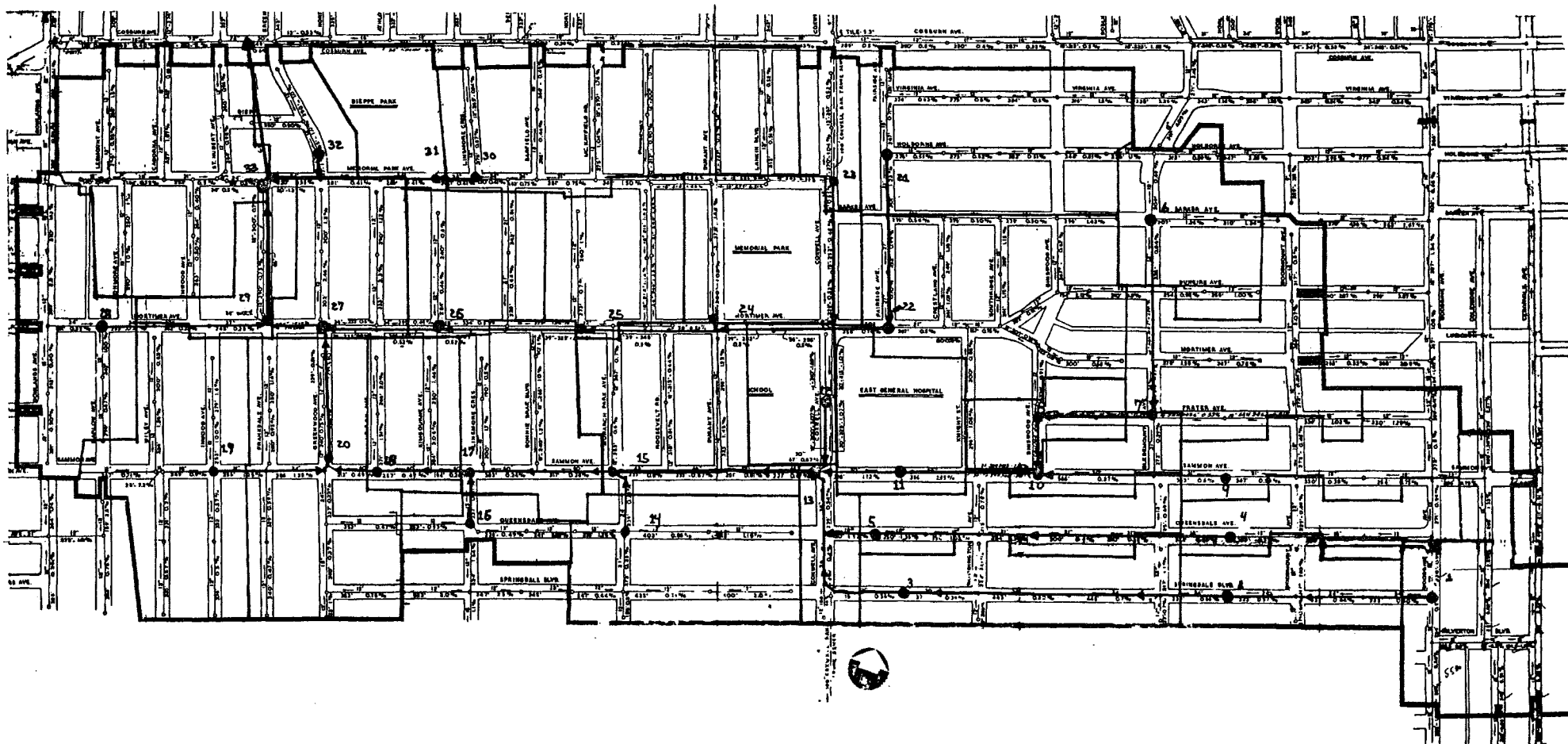


Figure 11 Pipe Network and Area Designation for the Catchment Test Area (96)



<div> <div> <div>1</div> <div>2</div> <div>3</div> <div>4</div> <div>5</div> <div>6</div> <div>7</div> <div>8</div> <div>9</div> <div>10</div> <div>11</div> <div>12</div> <div>13</div> <div>14</div> <div>15</div> <div>16</div> <div>17</div> <div>18</div> <div>19</div> <div>20</div> <div>21</div> <div>22</div> <div>23</div> <div>24</div> <div>25</div> <div>26</div> <div>27</div> <div>28</div> <div>29</div> <div>30</div> <div>31</div> <div>32</div> </div> </div>		<div>APPROVED</div> <div> <div> <div>DESIGNED BY</div> <div>DATE</div> </div> </div>			<div> <div> <div> <div></div> <div>Consulting Engineers &amp; Planners</div> </div> </div> </div>	<div> <div> <div> <div>DATE</div> <div>BY</div> <div>CHKD</div> <div>APPD</div> </div> <div> <div>11</div> <div>12</div> <div>13</div> <div>14</div> </div> </div> </div>	<div> <div>ENVIRONMENT CANADA</div> <div>"AS BUILT" SEWER PLAN</div> </div>	<div> <div>9</div> </div>
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Figure 12 Drainage Plan for Test Site - Toronto, Canada (104)

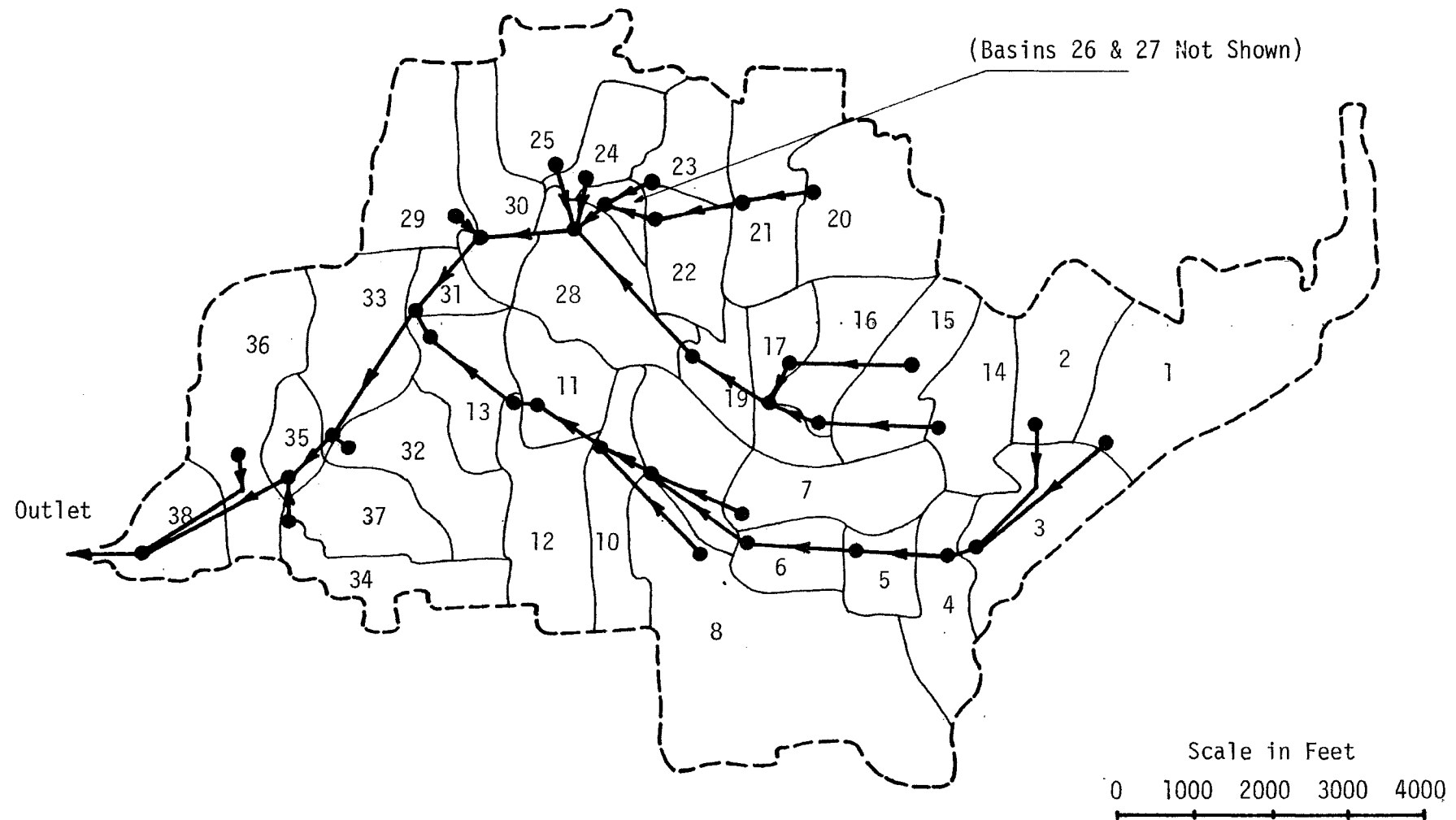
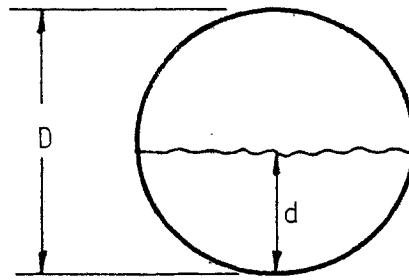


Figure 13 Schematic of Bloody Run Storm Sewer Network - Cincinnati, Ohio (110)



$d$  = depth of flow  
 $D$  = diameter of pipe

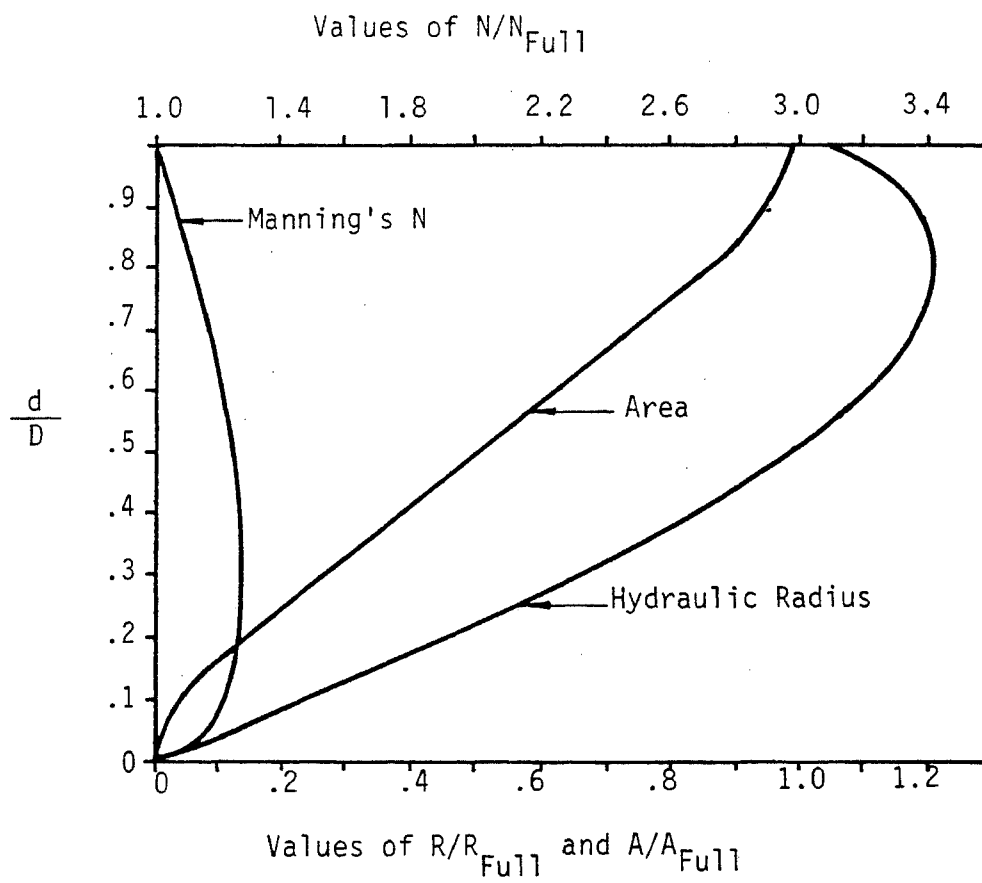
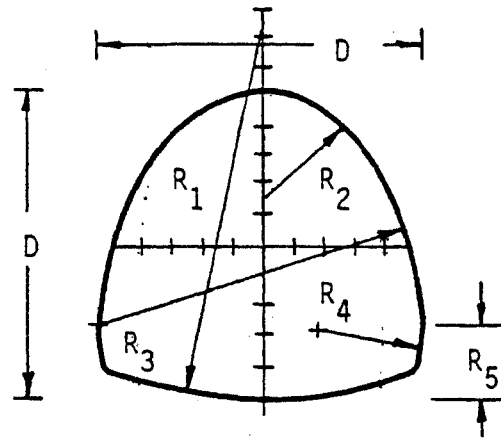


Figure 14. Characteristics of the Circular Pipe Section (4)



$D$  = Diameter of semielliptical conduit  
 (R values as shown)

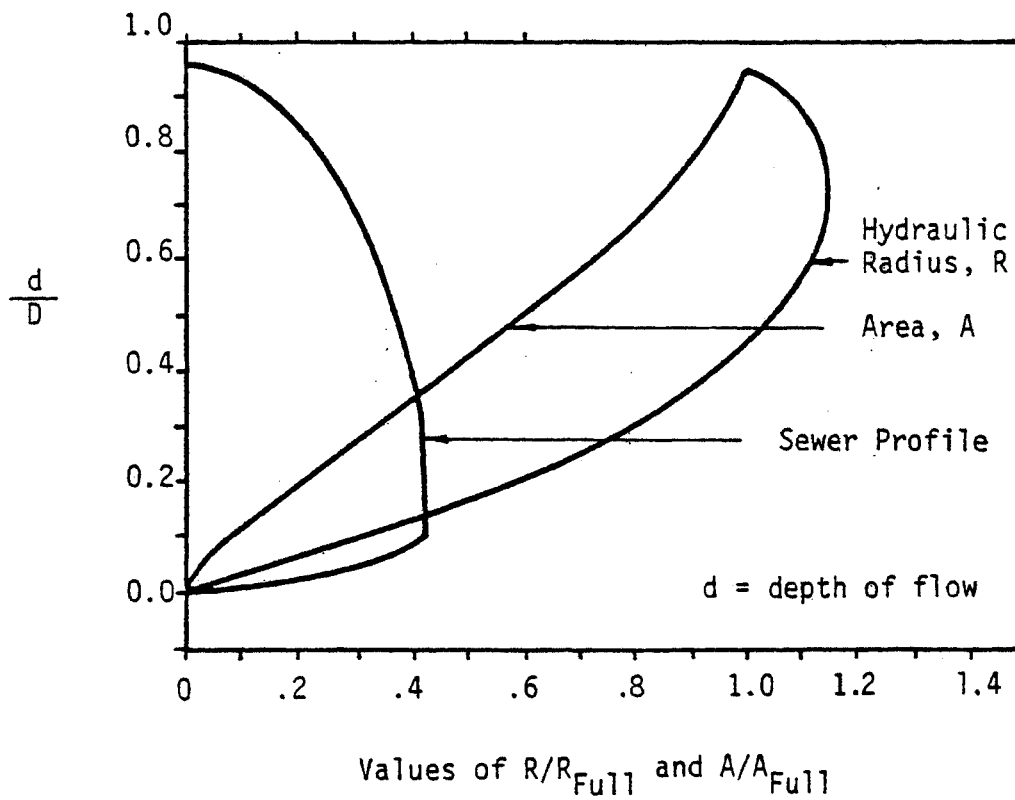
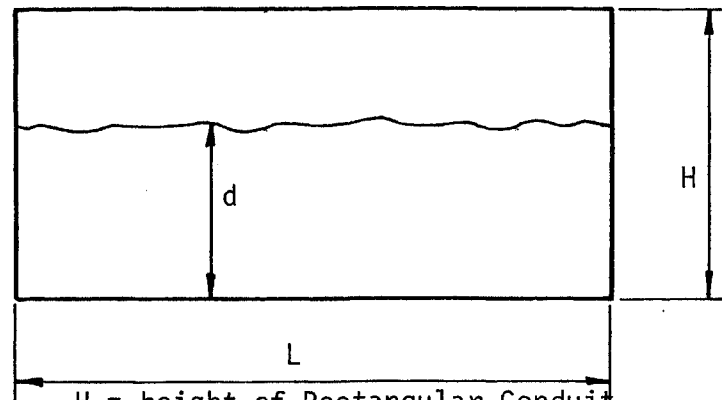


Figure 15 Characteristics of the Semielliptical Conduit (127)



$H$  = height of Rectangular Conduit

$L$  = width of Rectangular Conduit

$d$  = depth of flow

$N$  = Manning's  $n$

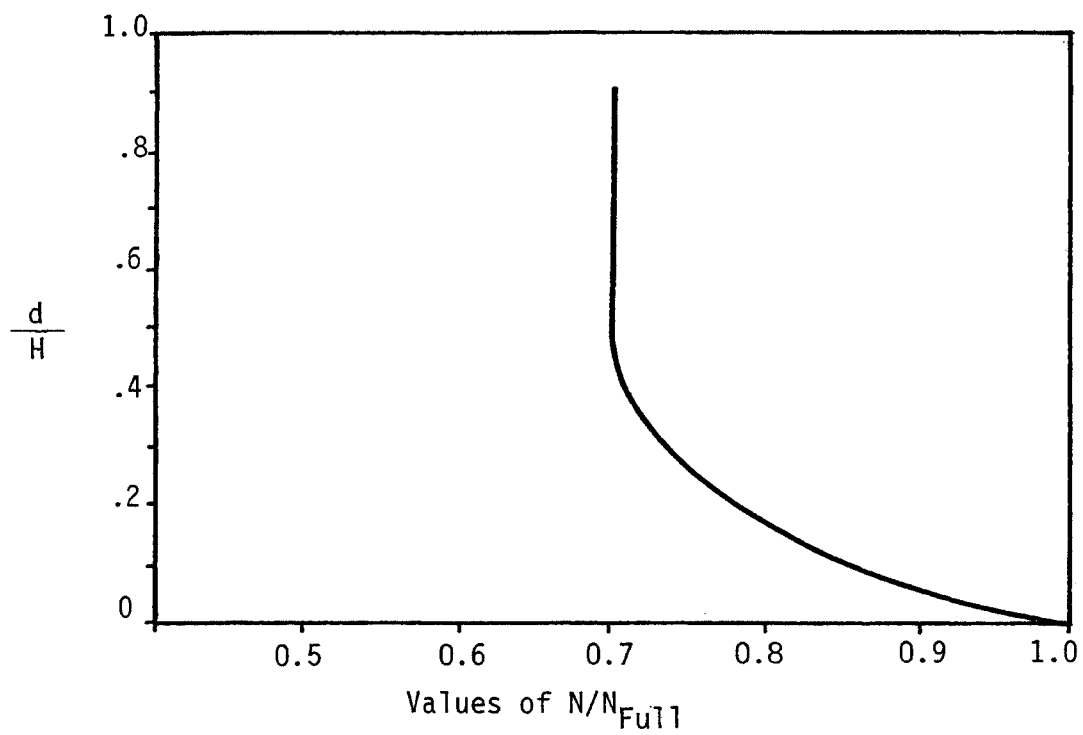


Figure 16 Characteristics of the Rectangular Conduit (82)



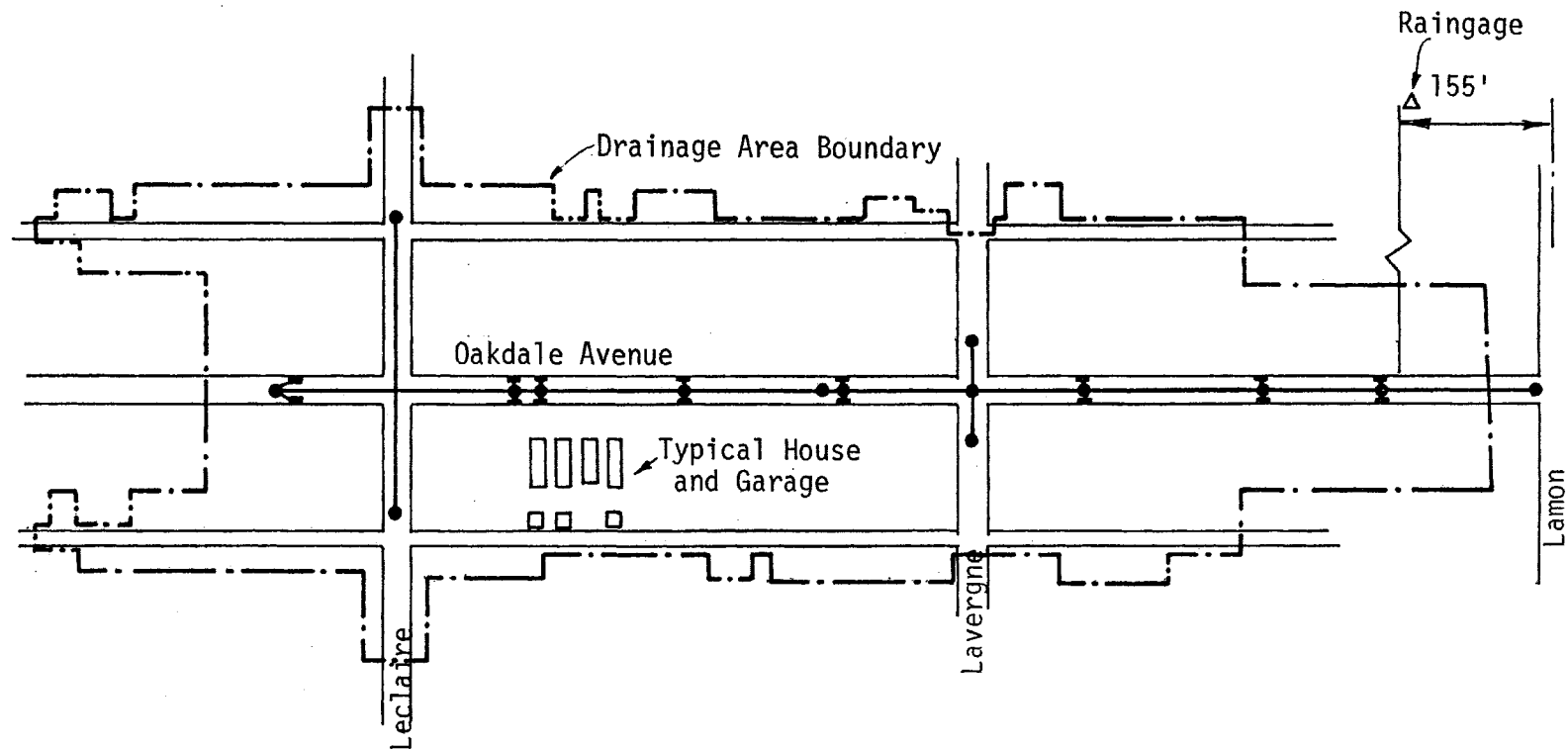


Figure 17 General Plan of Oakdale Avenue Drainage Basin, Chicago (19)

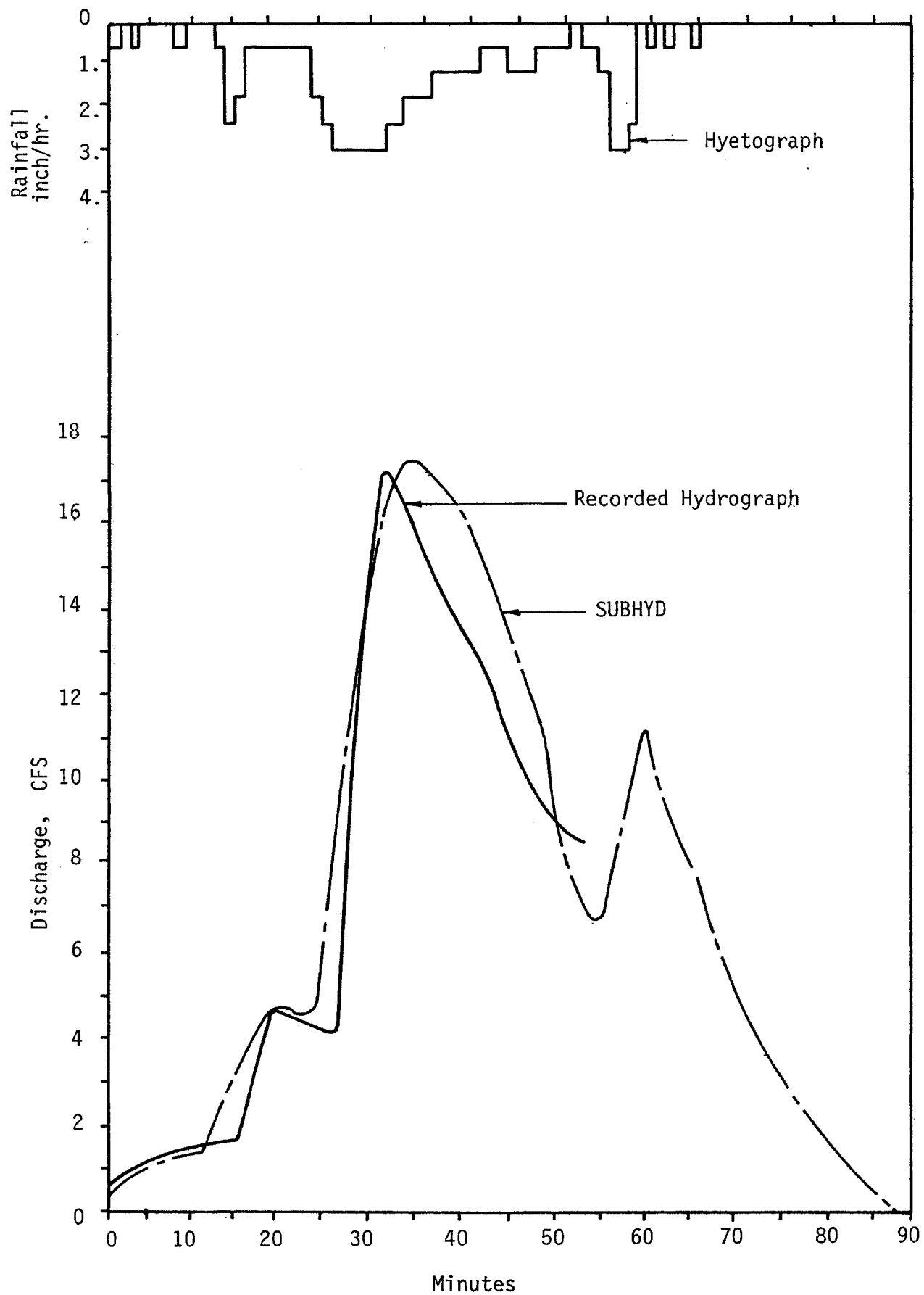


Figure 18 Results from Oakdale Avenue Drainage Basin, Chicago  
Storm of July 2, 1960 (19)

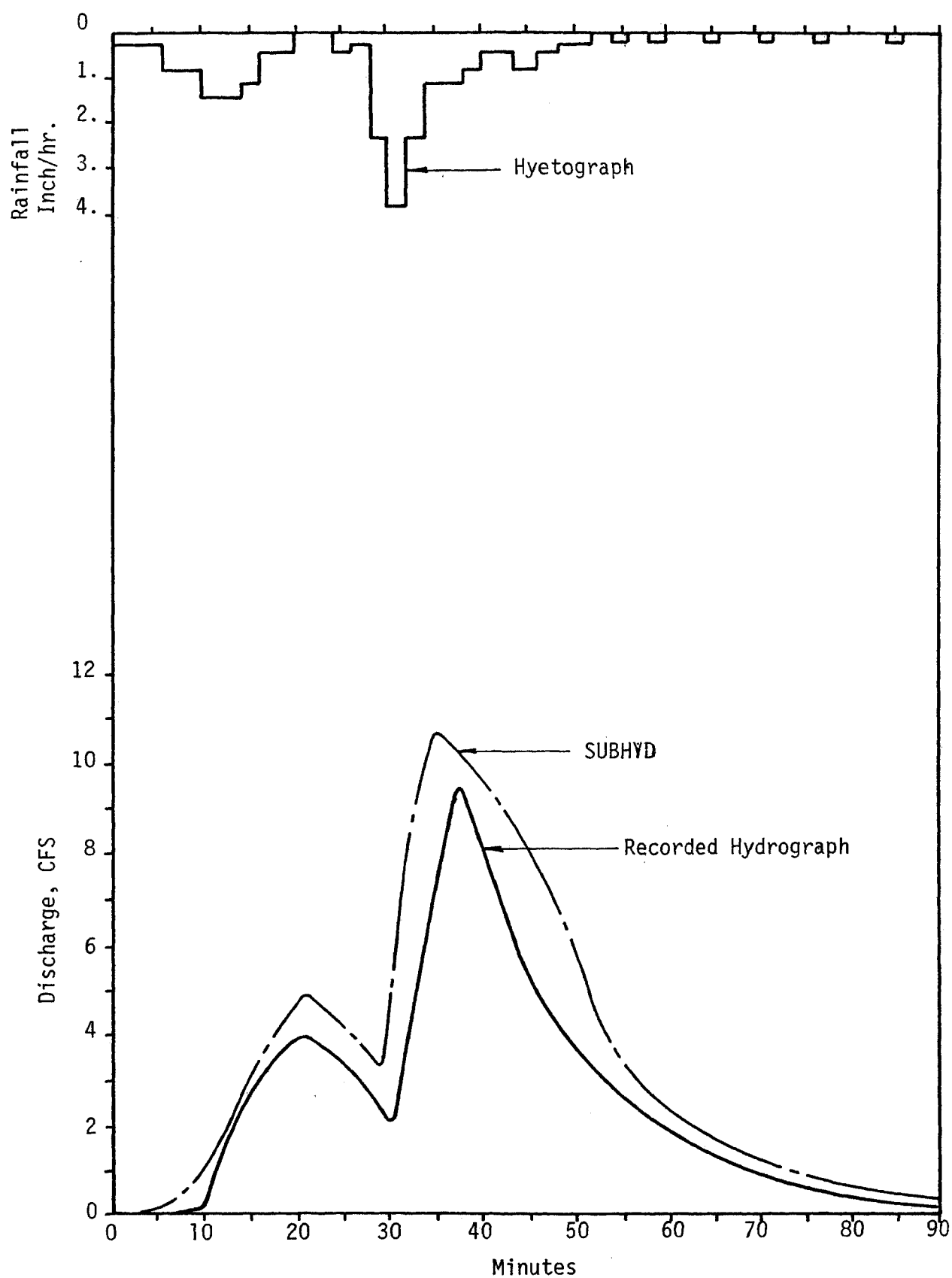


Figure 19 Results from Oakdale Avenue Drainage Basin, Chicago  
Storm of July 7, 1964 (19)



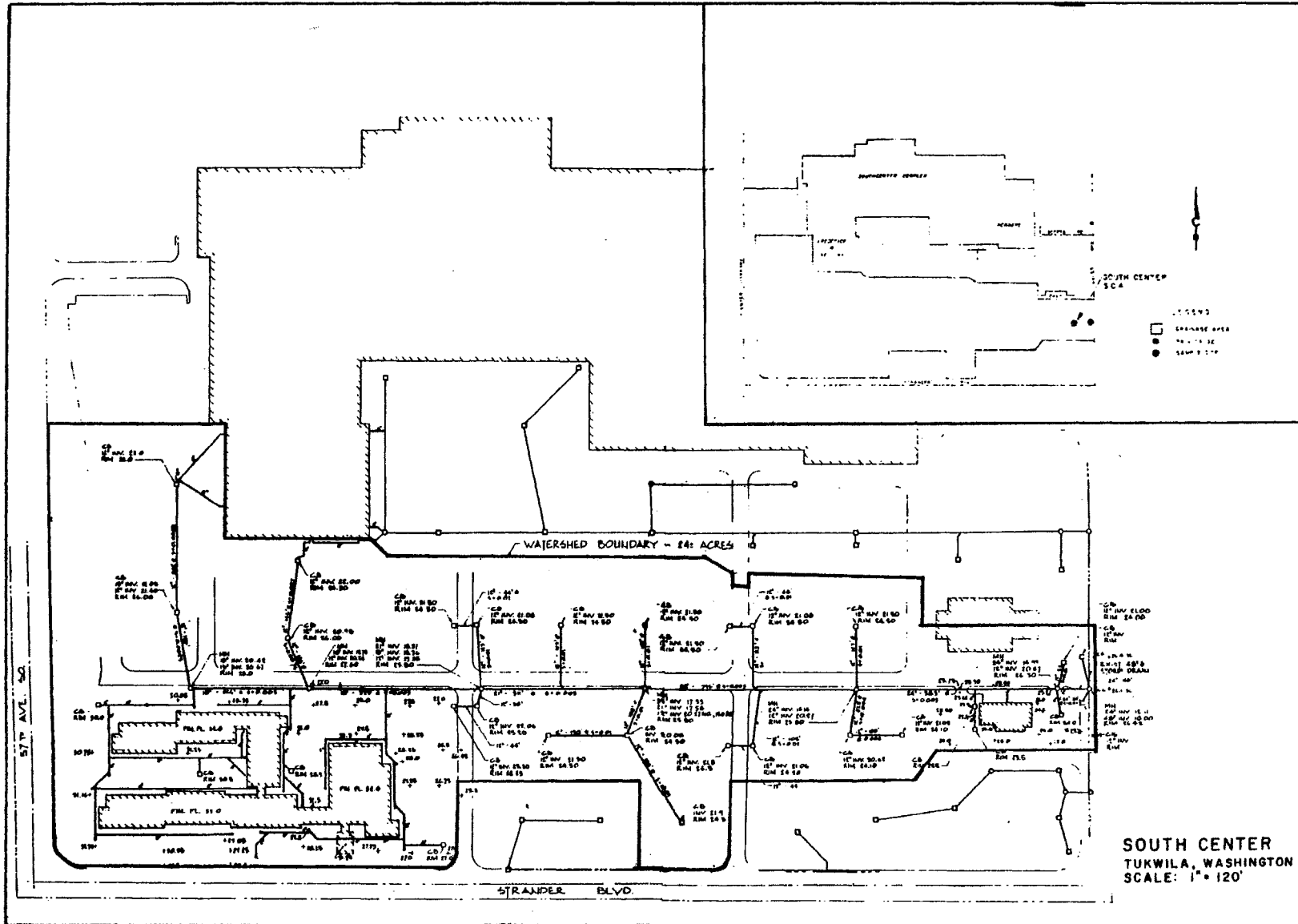
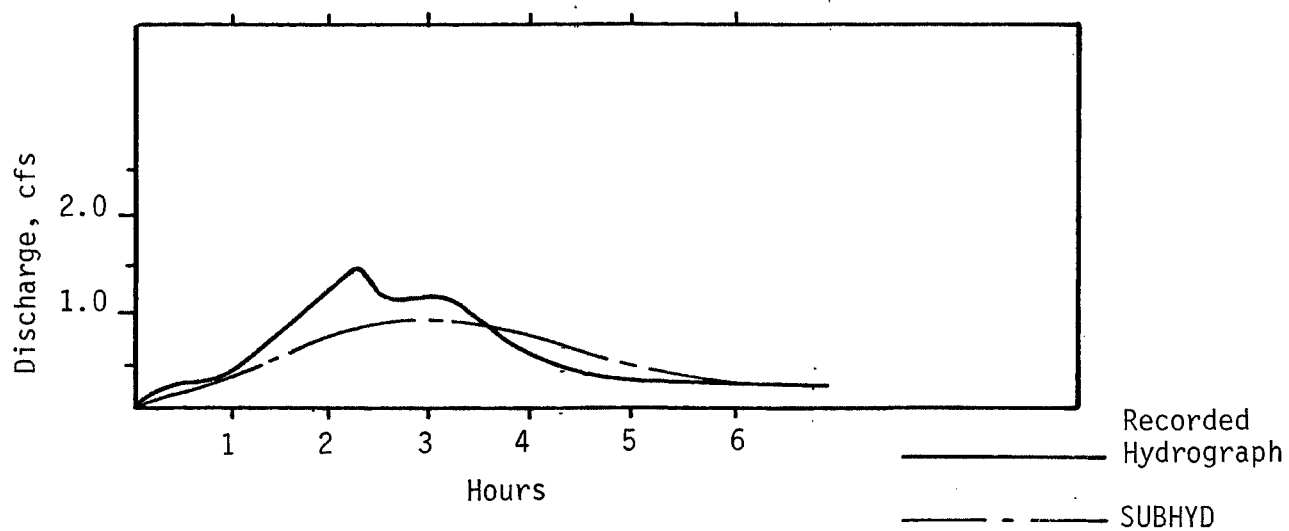
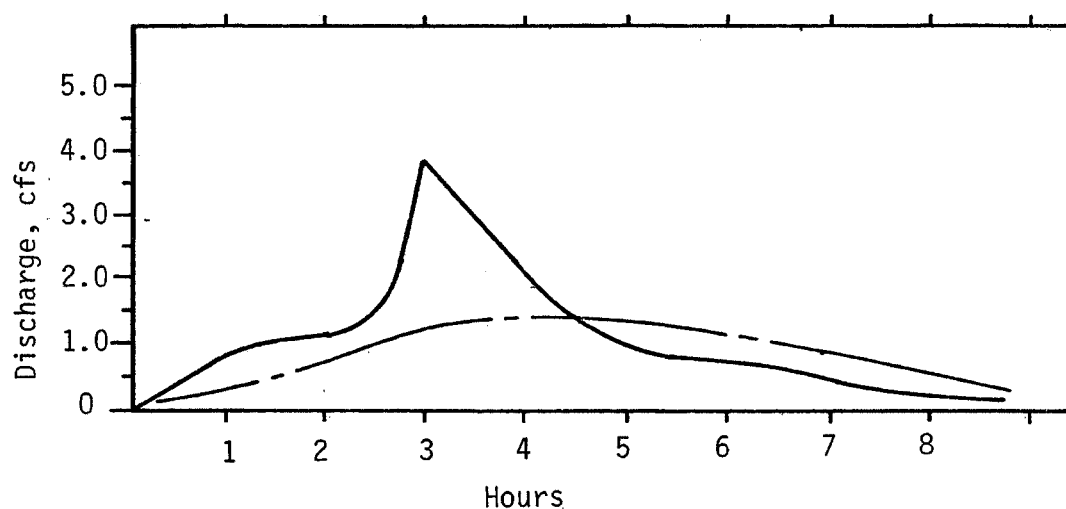


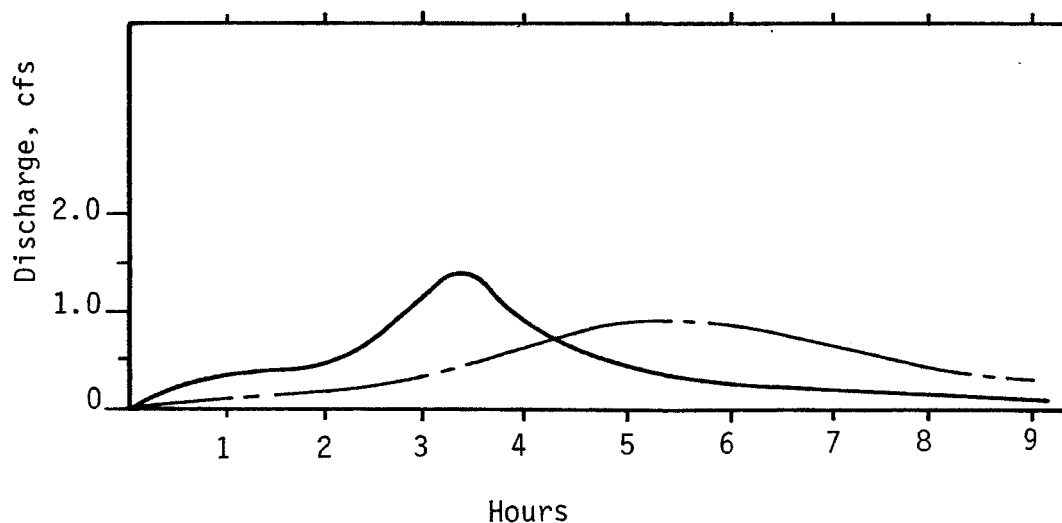
Figure 21 General Plan of South Center Drainage Basin-Tukwila, Washington (126)



a. Storm of November 16, 1975, South Seattle



b. Storm of December 7, 1975, South Seattle



c. Storm of February 15, 1975, South Center, Tukwila

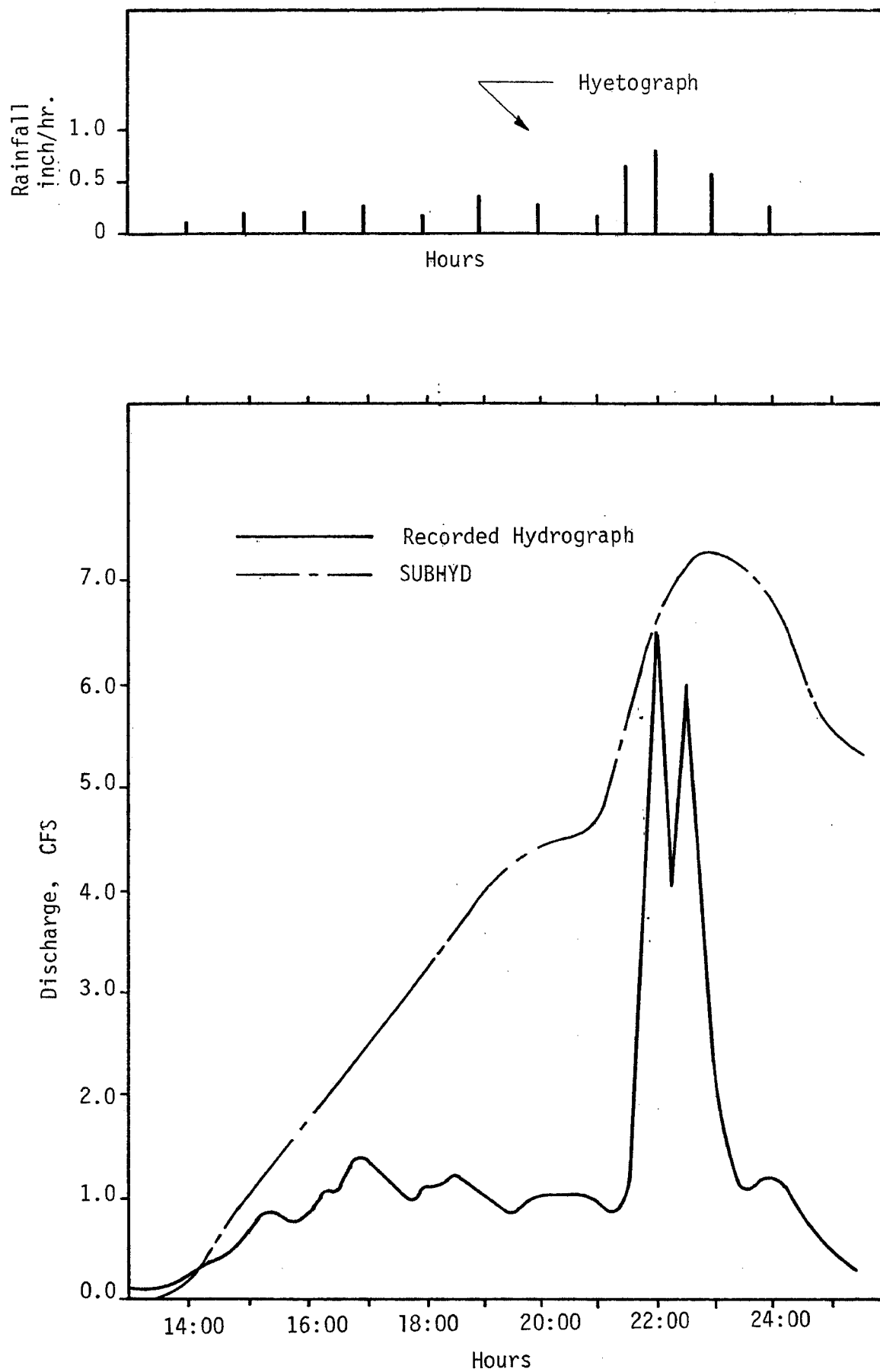


Figure 23 Results from South Center, Tukwila, Washington  
September 23, 1974 (126)

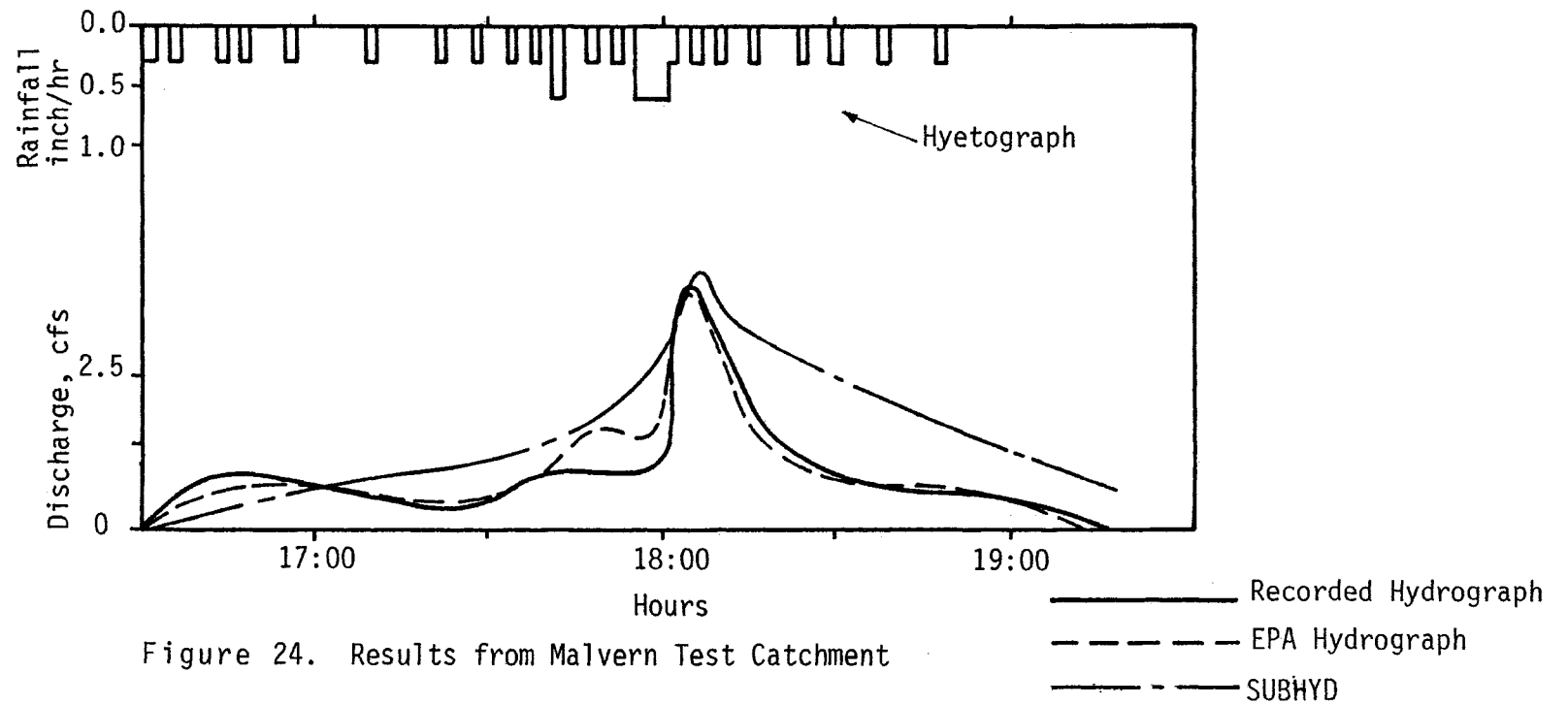


Figure 24. Results from Malvern Test Catchment

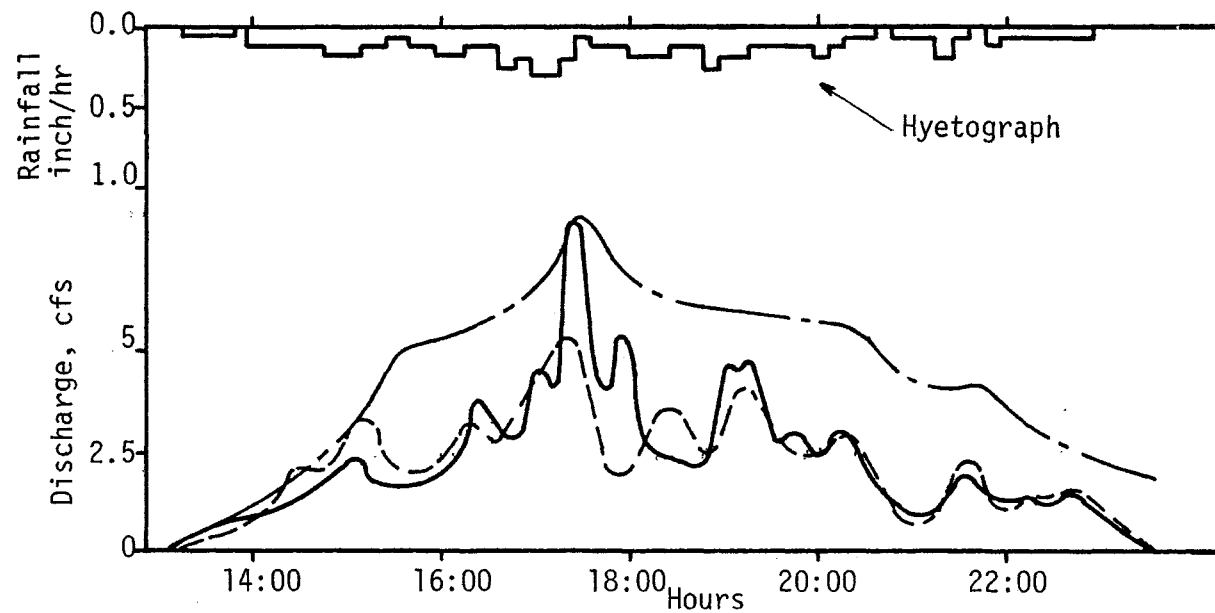


Figure 25 Results from Malvern Urban Test Catchment, Burlington, Ontario (96)



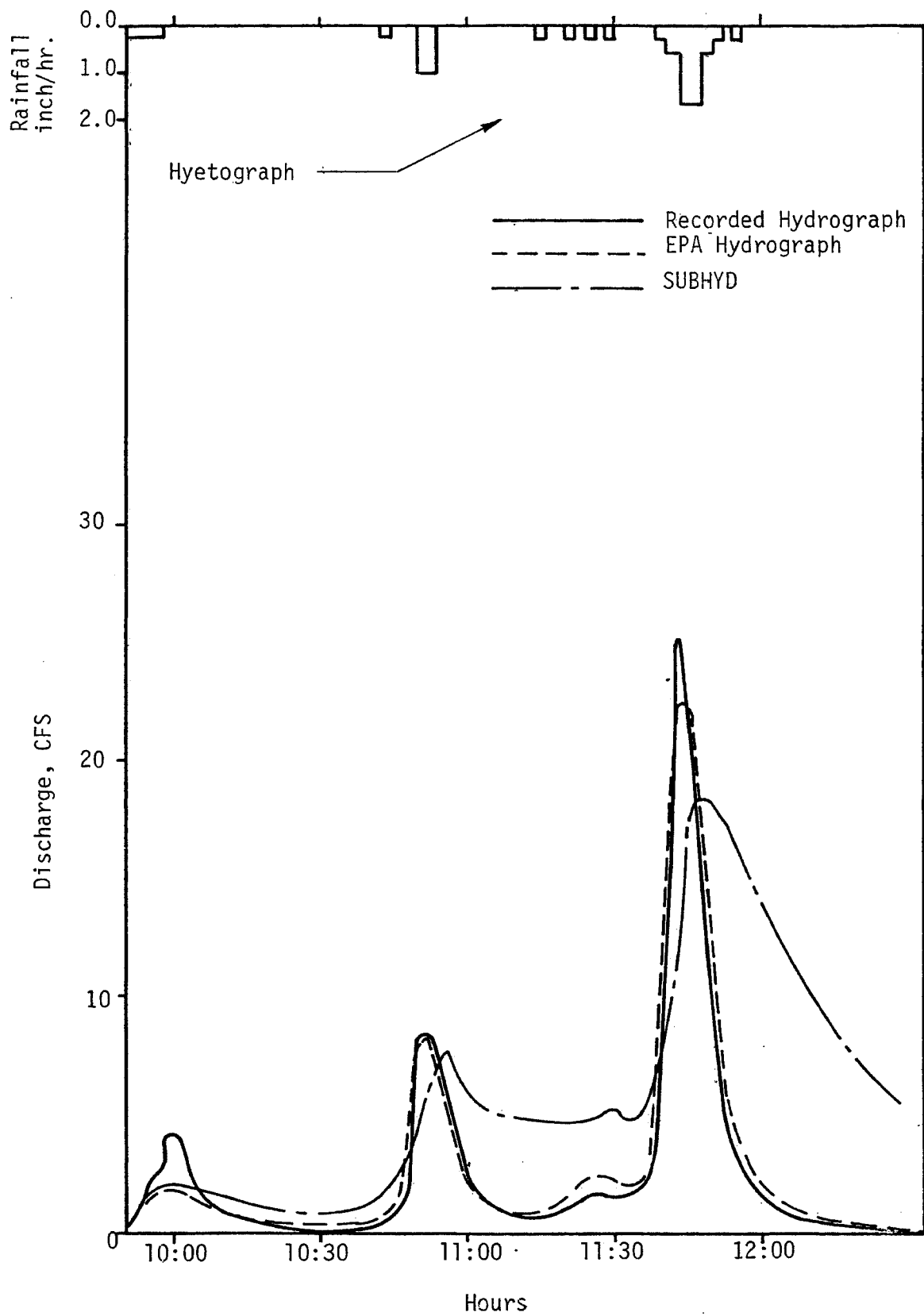


Figure 26 Results from Malvern Urban Test Catchment - Burlington, Ontario (96)

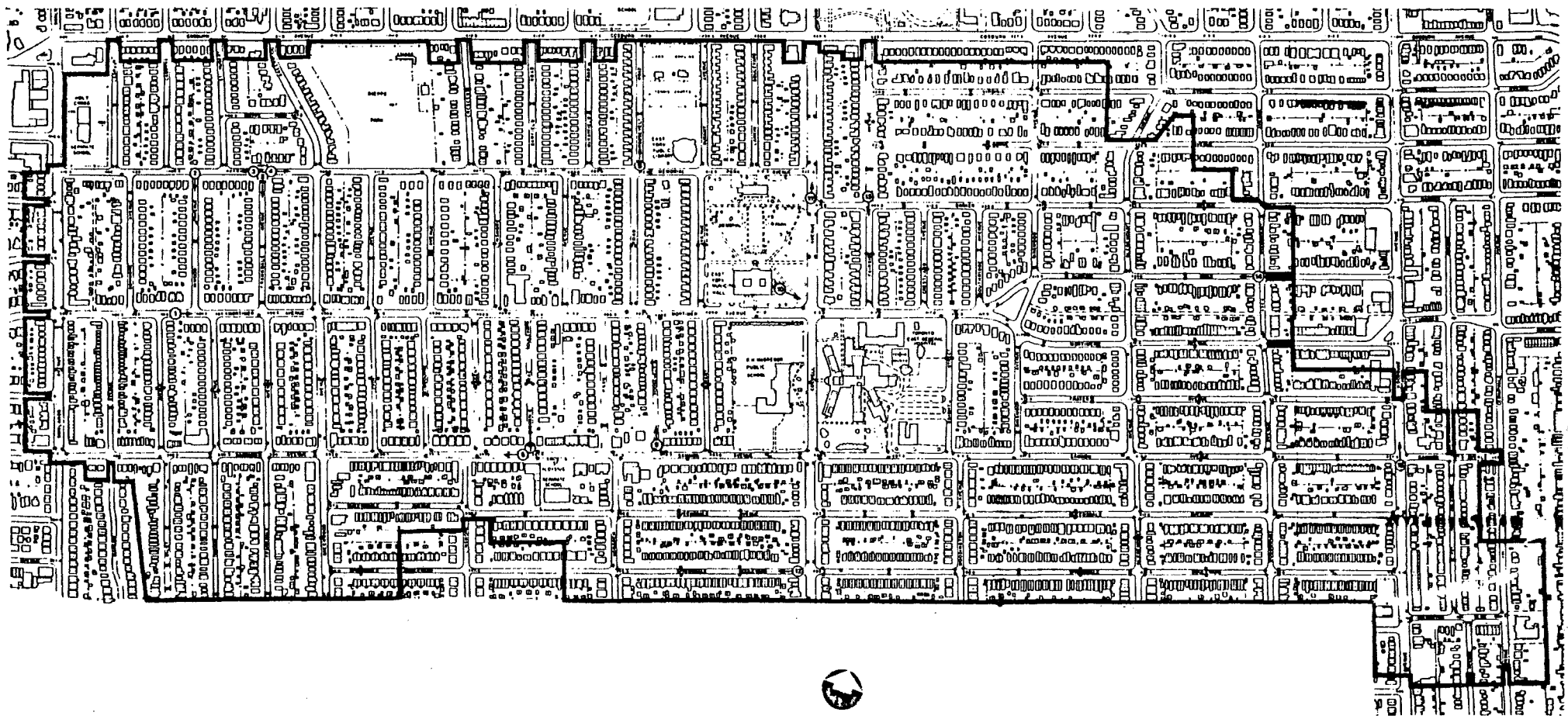


Figure 27 Topographical Map of Urban Test Site for Toronto, Canada (104)

LEGEND  
 ——— BOUNDARY OF URBAN TEST AREA  
 + CATCH BASIN  
 ⊙ LOCATION AND DIRECTION OF PHOTOGRAPHY

		APPROVED			 Consulting Engineers & Planners		ENVIRONMENT CANADA TOPOGRAPHICAL MAP 2
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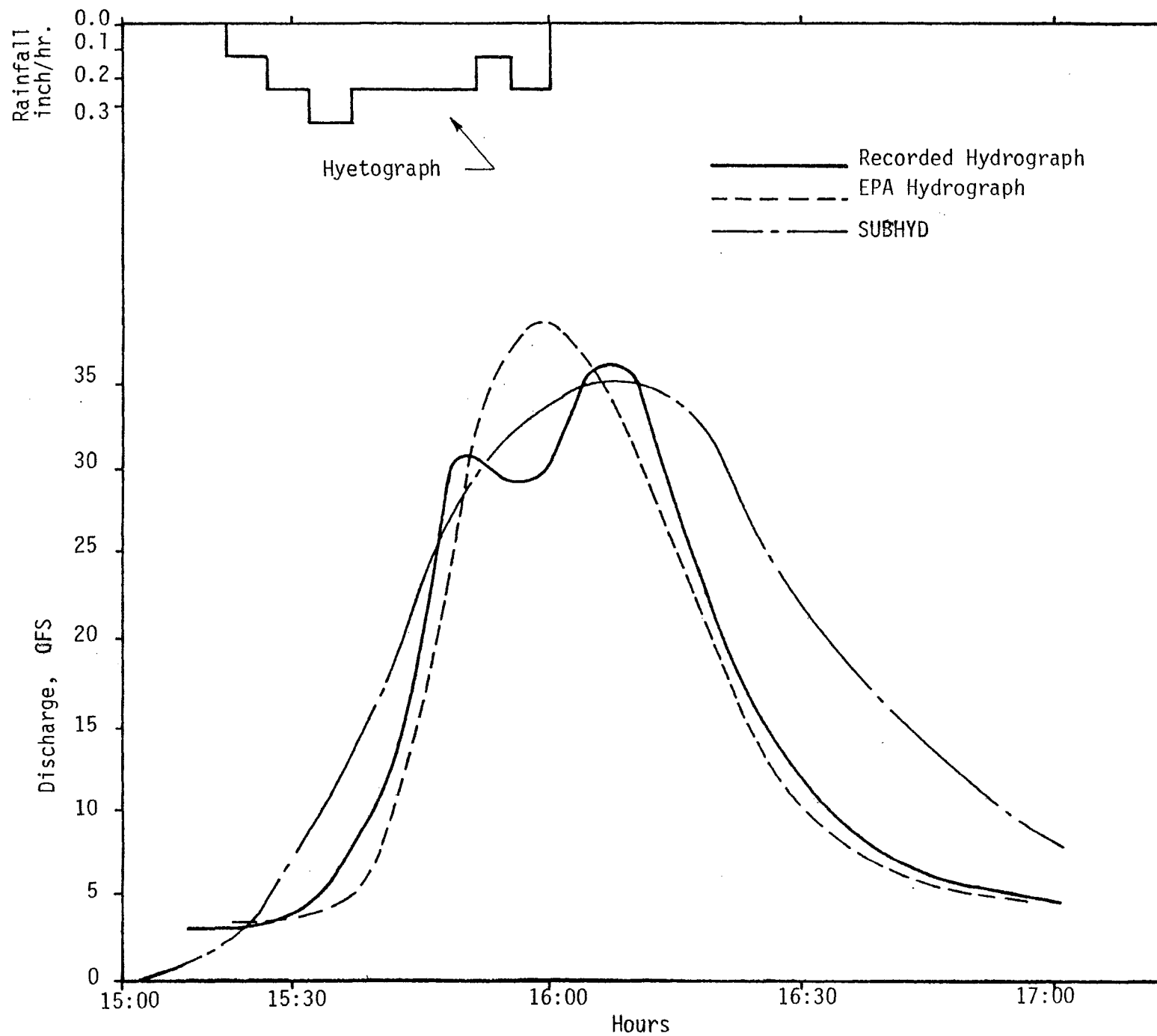


Figure 28 Results from Toronto, Canada — May 11, 1976 (104)

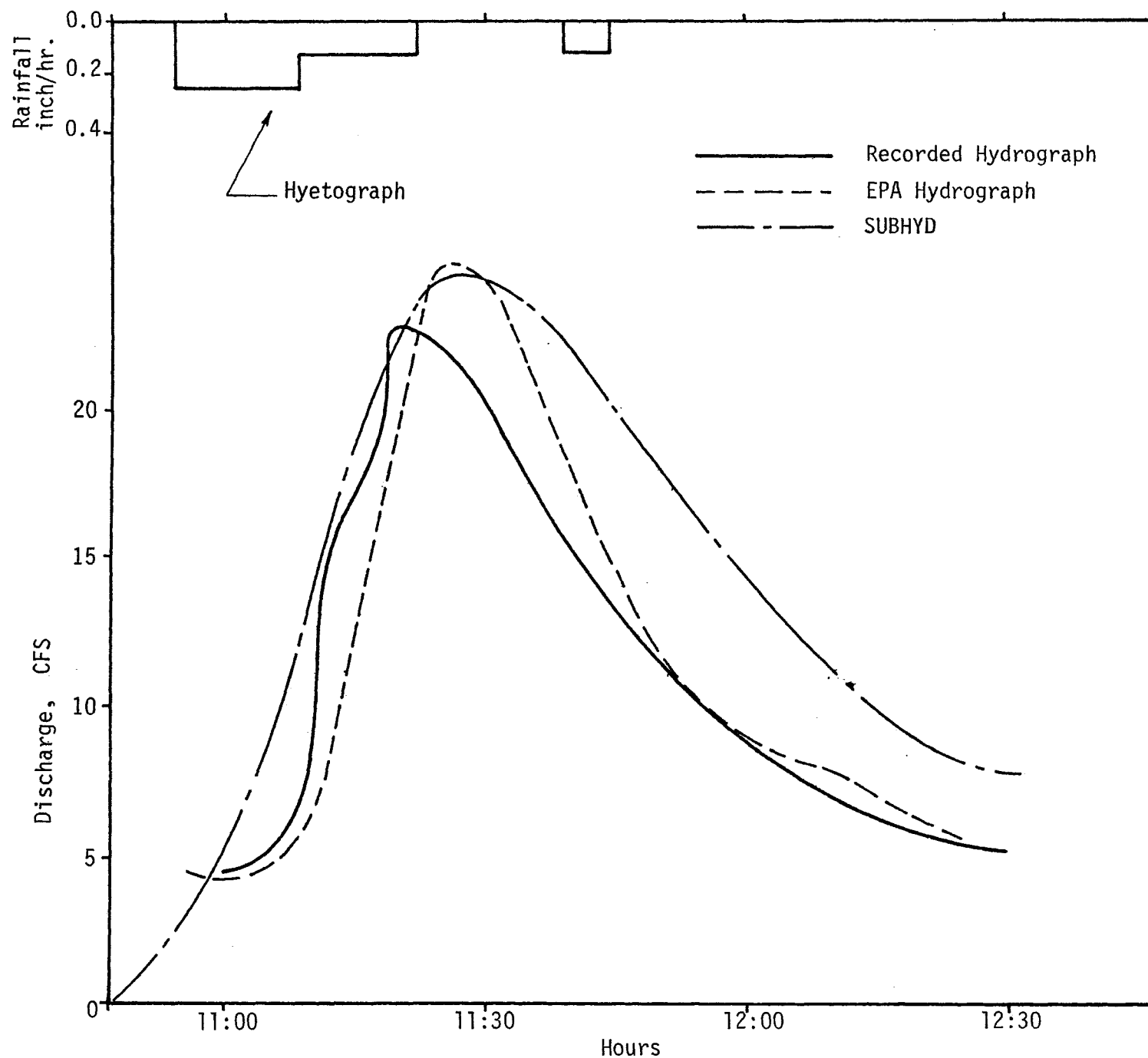


Figure 29 Results from Toronto, Canada-June 1, 1976 (104)

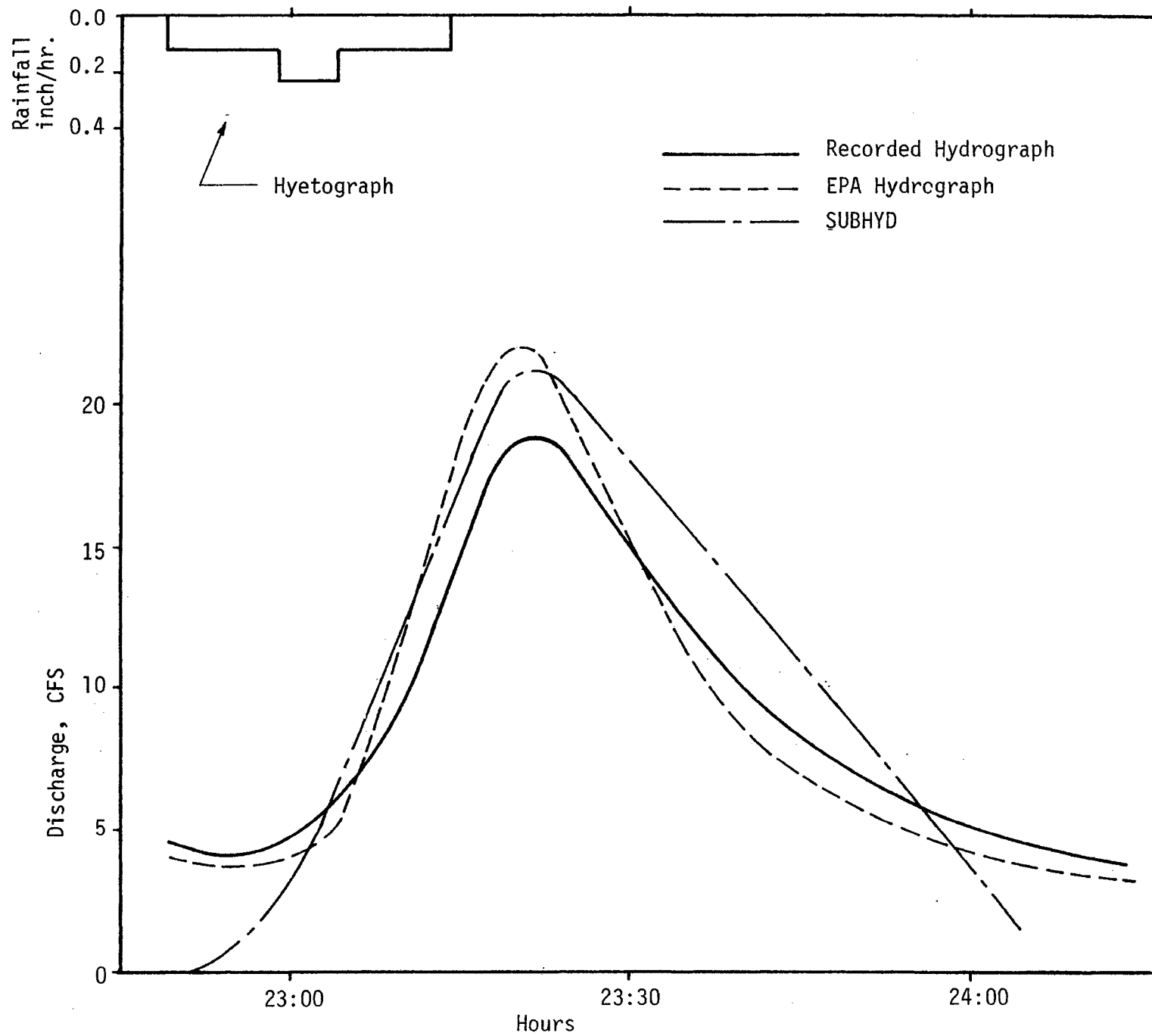


Figure 30 Results from Toronto, Canada - June 28, 1976 (104)

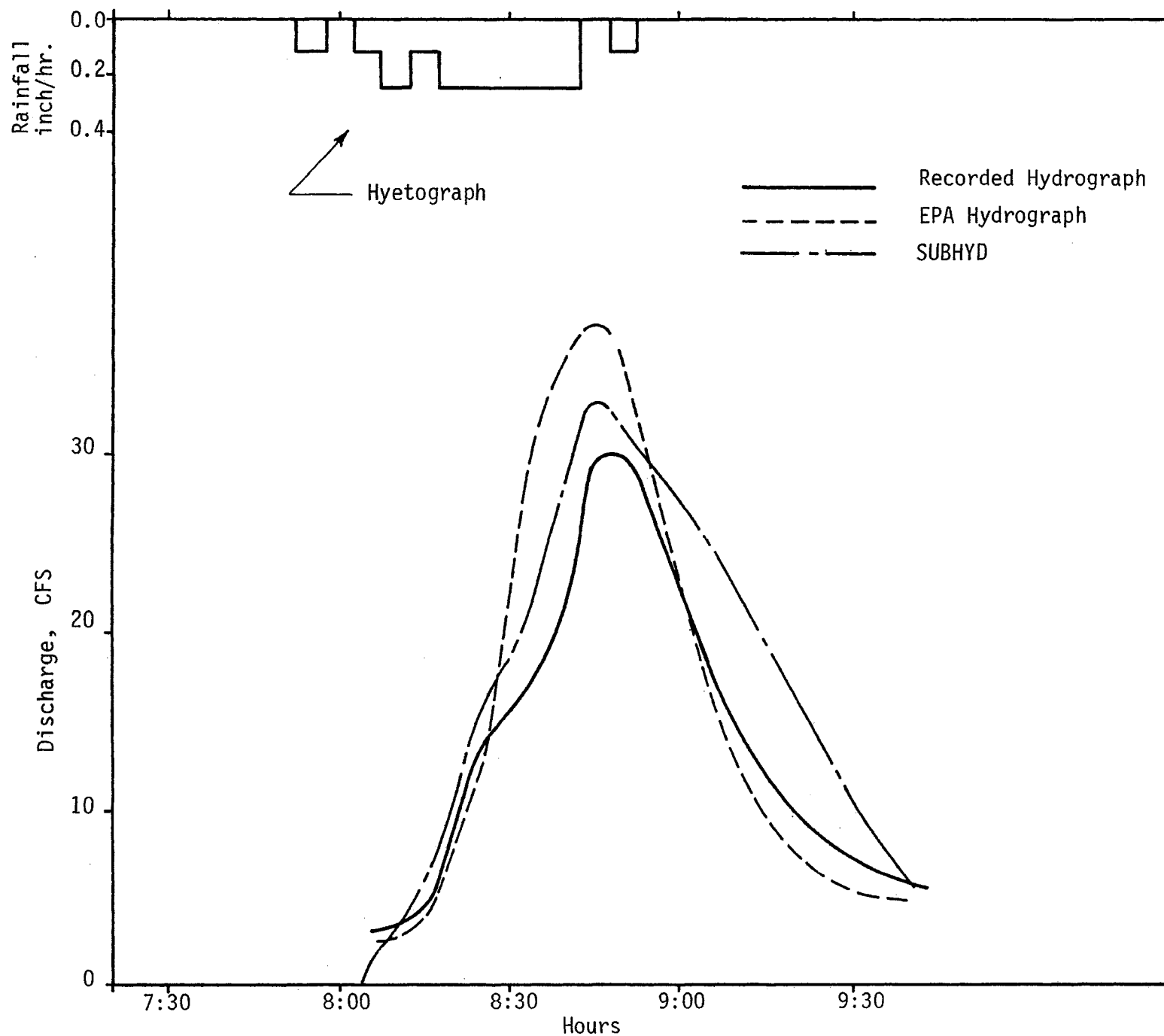


Figure 31 Results from Toronto, Canada - July 1, 1976 (104)

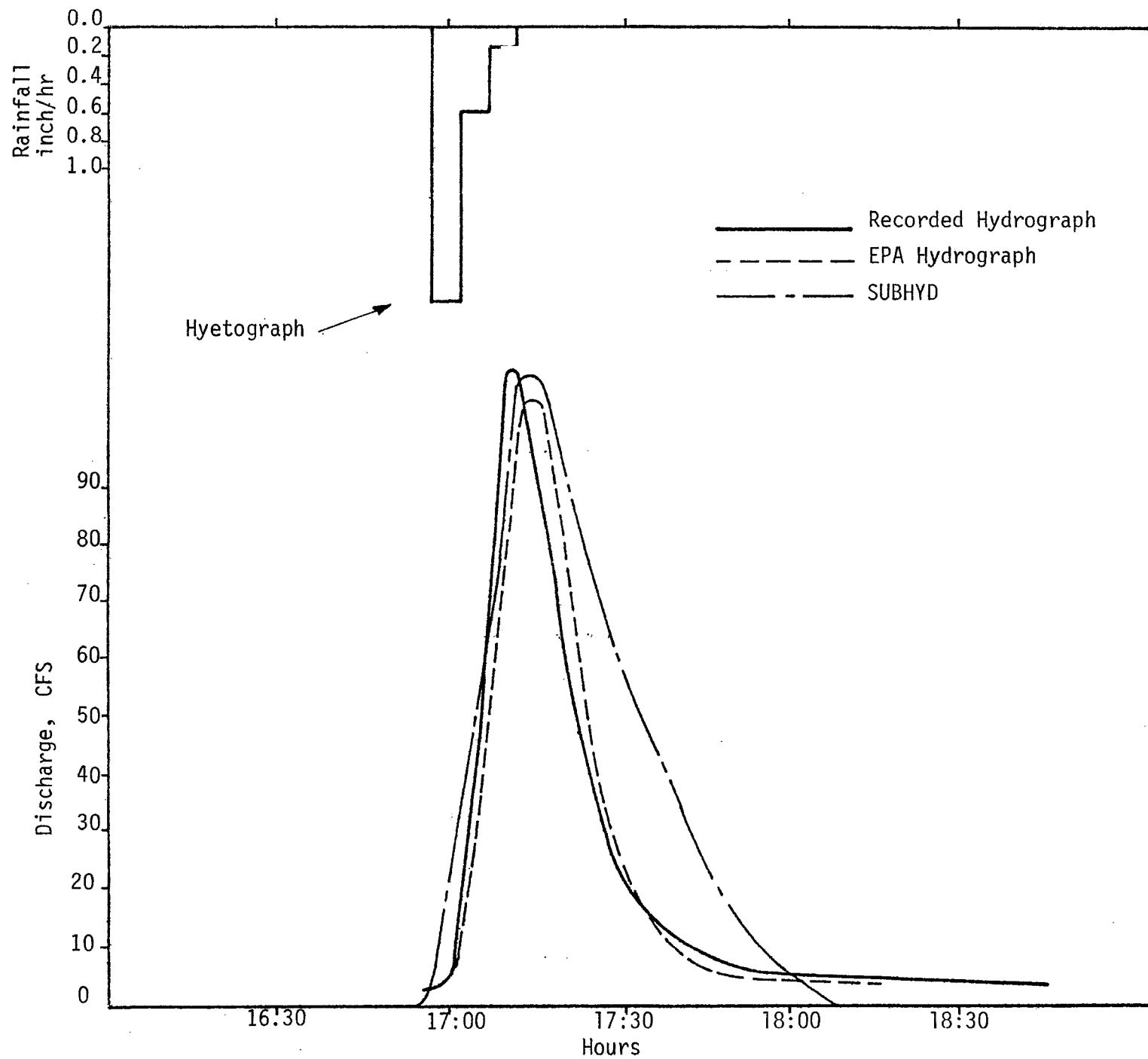


Figure 32 Results from Toronto, Canada - July 1, 1976 (104)

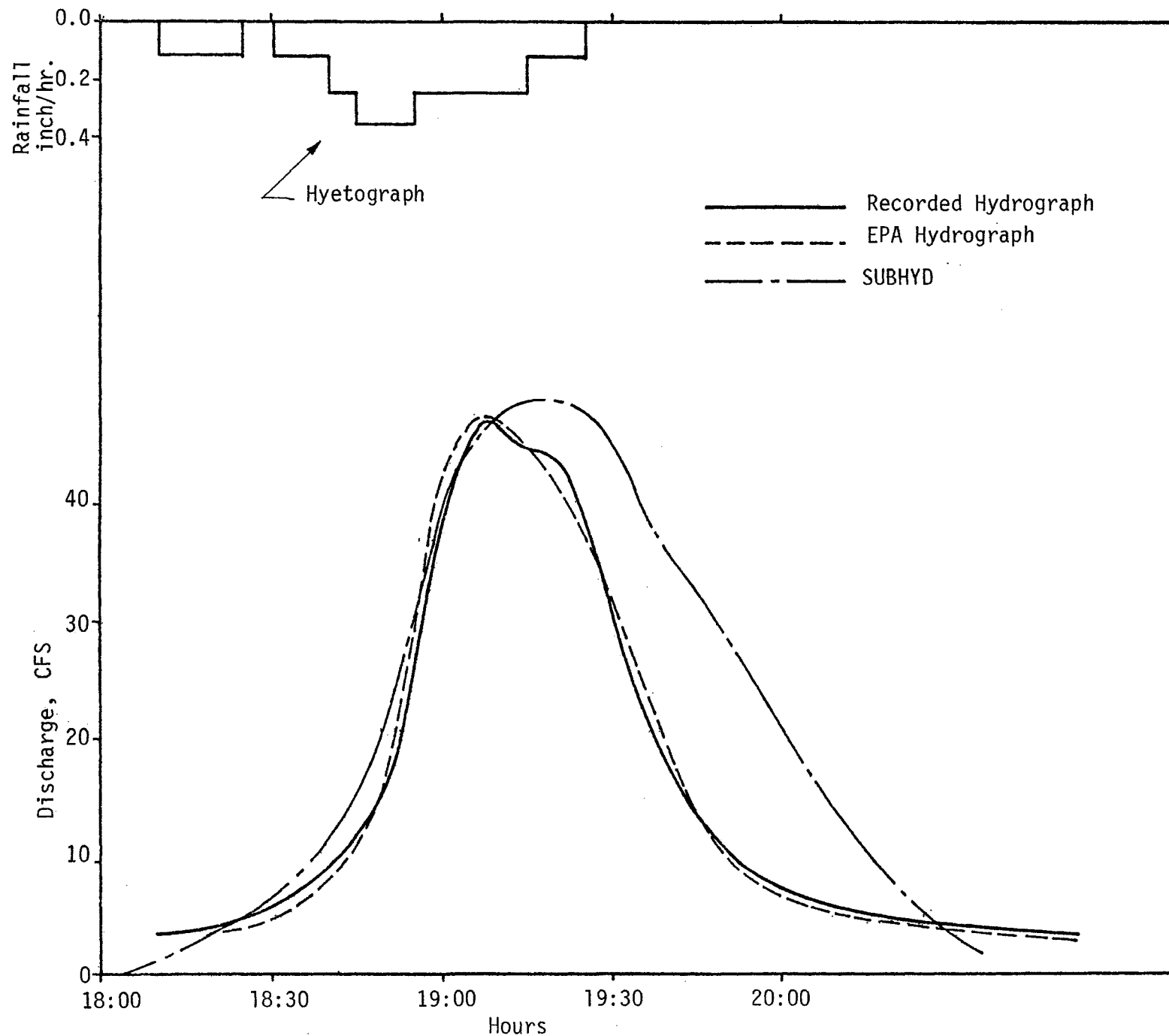


Figure 33 Results from Toronto, Canada - July 7, 1976 (104)



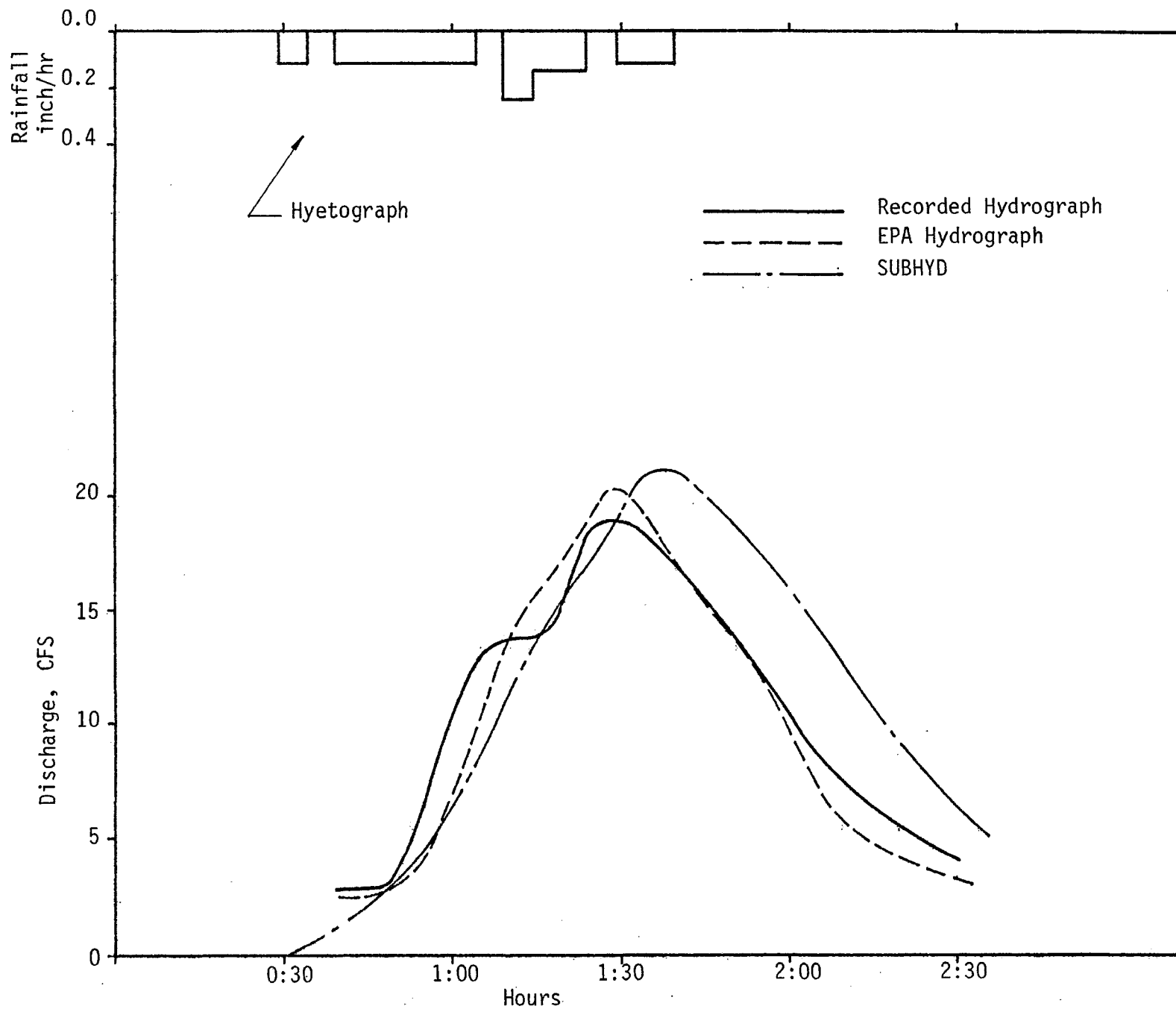


Figure 34 Results from Toronto, Canada - July 11, 1976 (104)

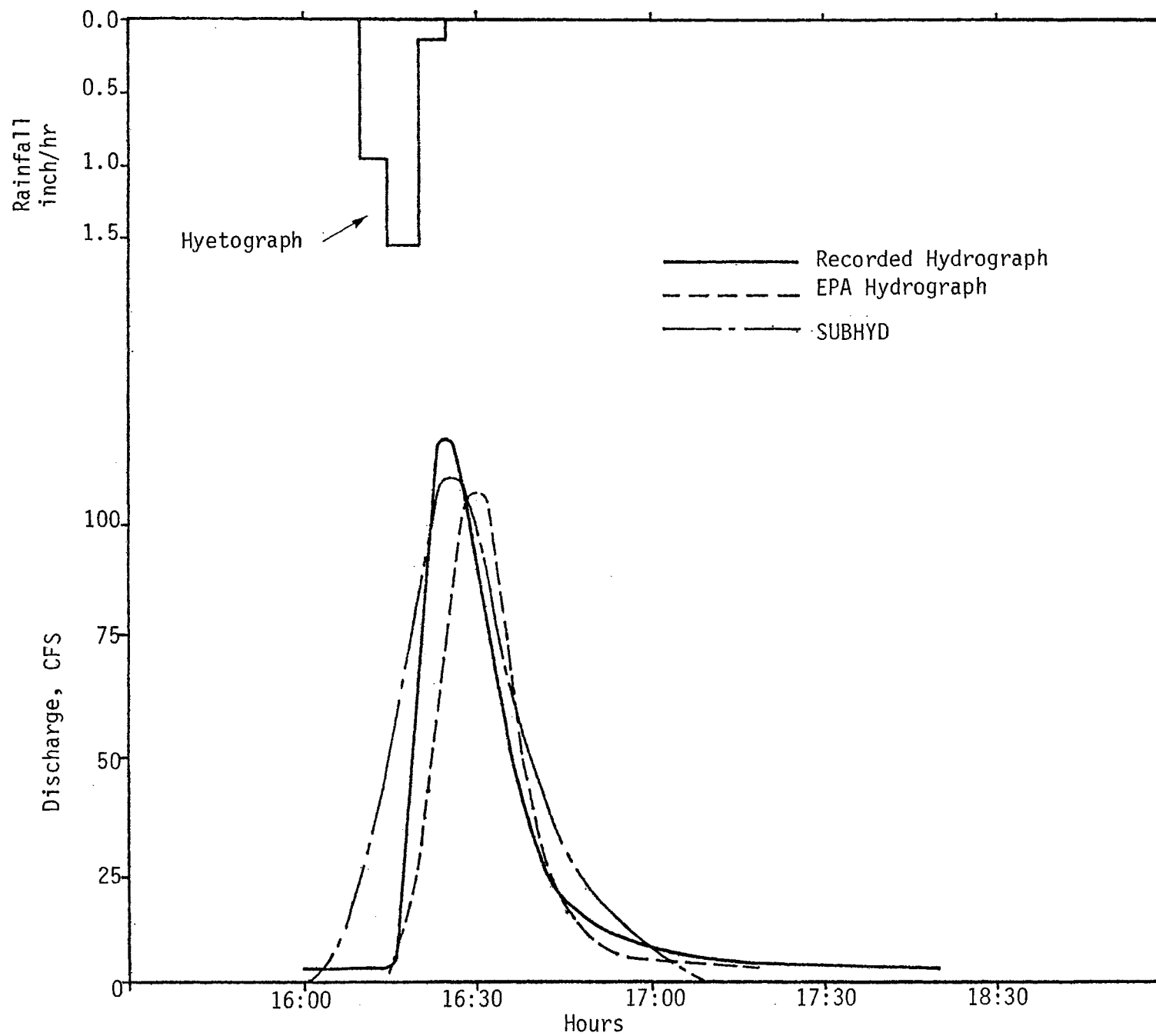


Figure 35 Results from Toronto, Canada - August 13, 1976 (104)

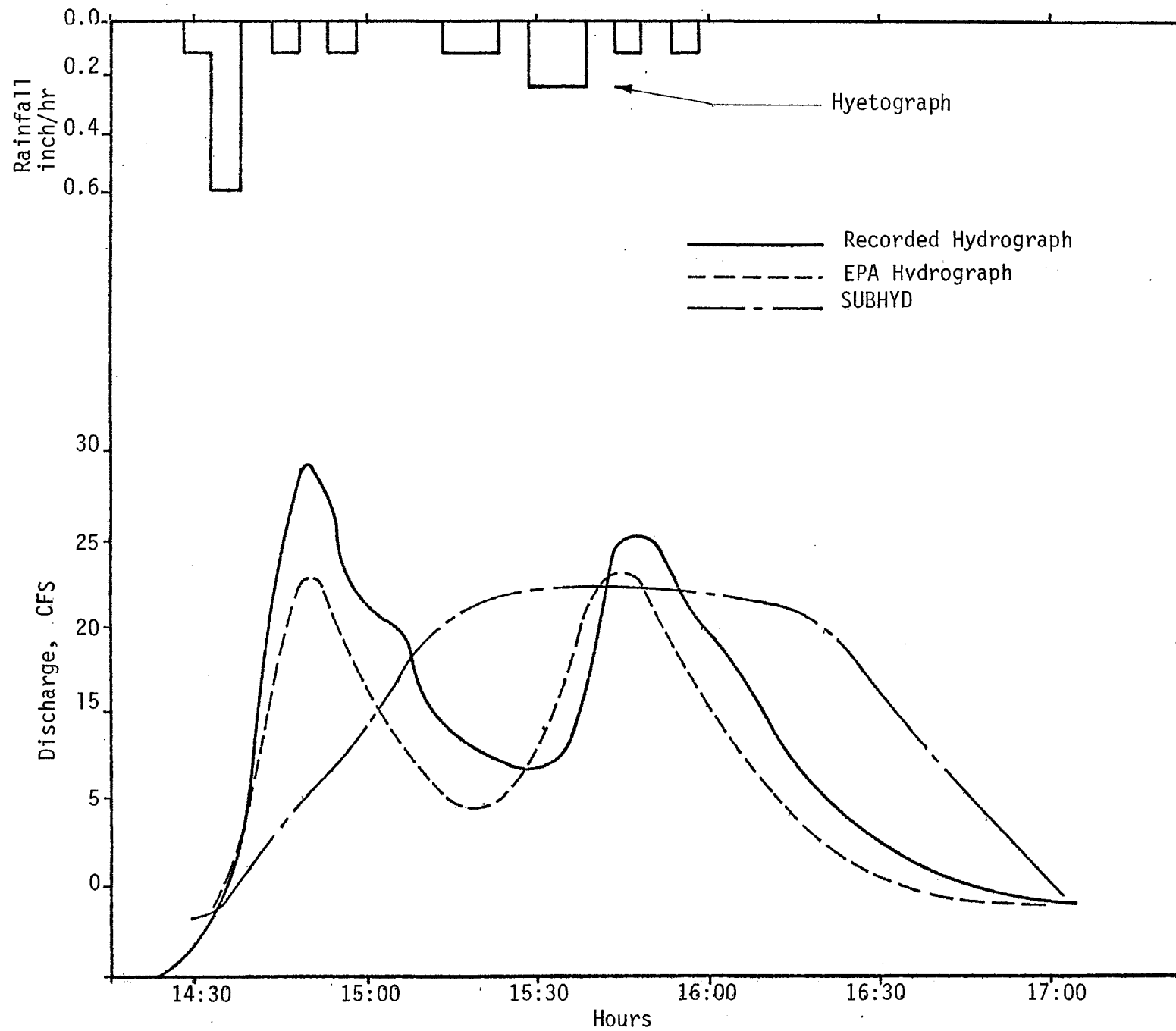


Figure 36 Results from Toronto, Canada - March 27, 1976 (104)

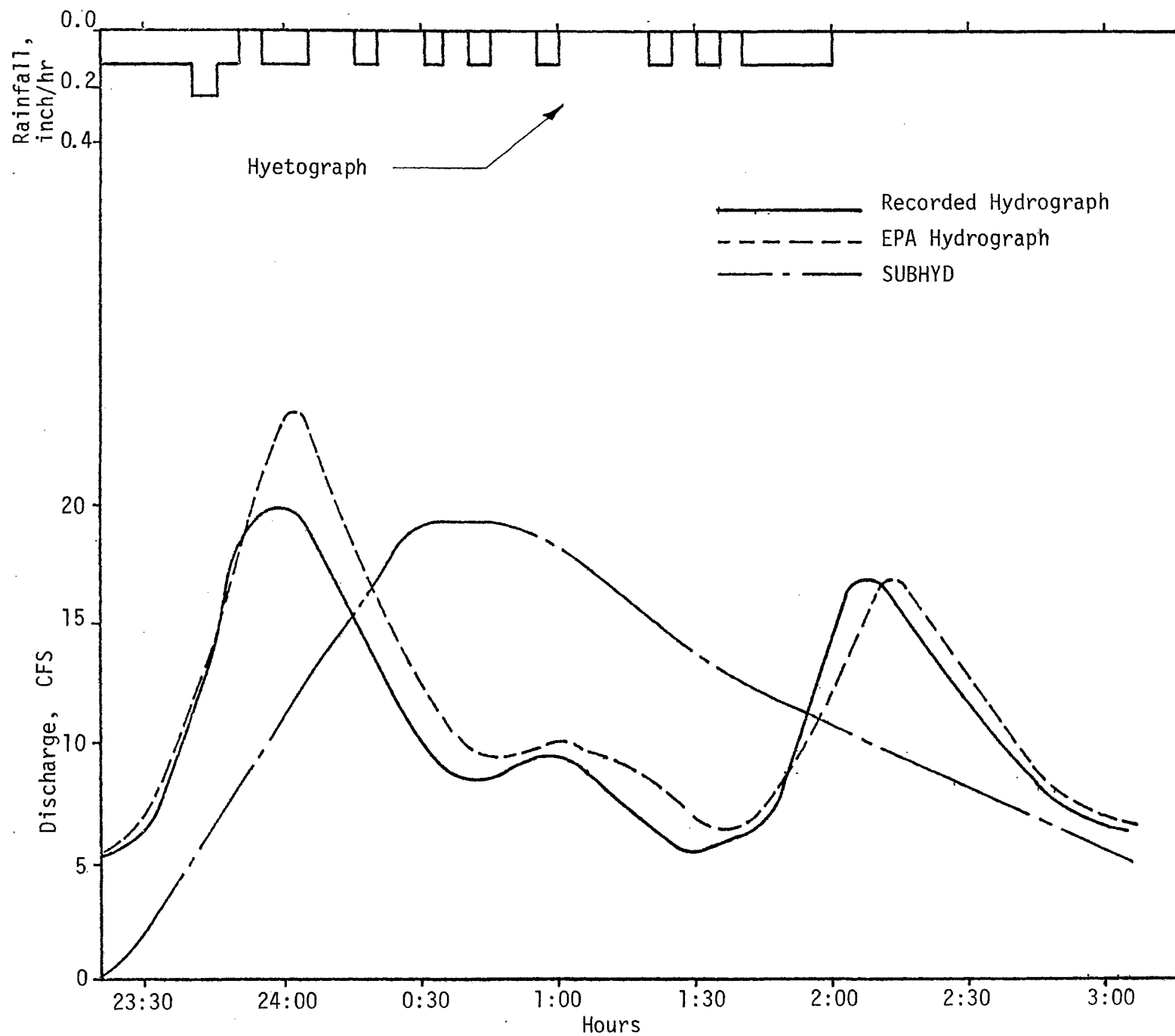


Figure 37 Results from Toronto, Canada April 25, 1976 (104)

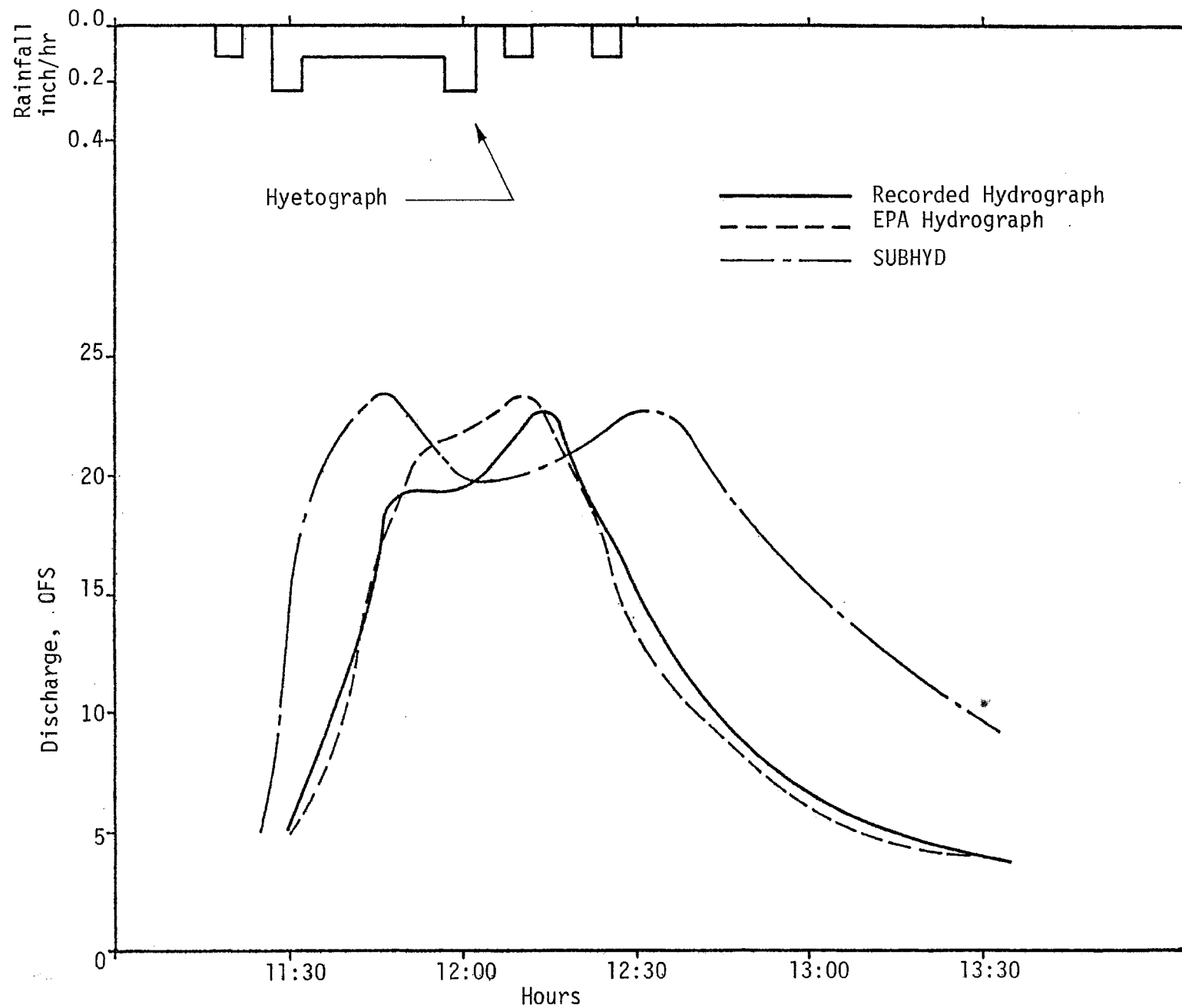


Figure 38 Results from Toronto, Canada July 2, 1976 (104)

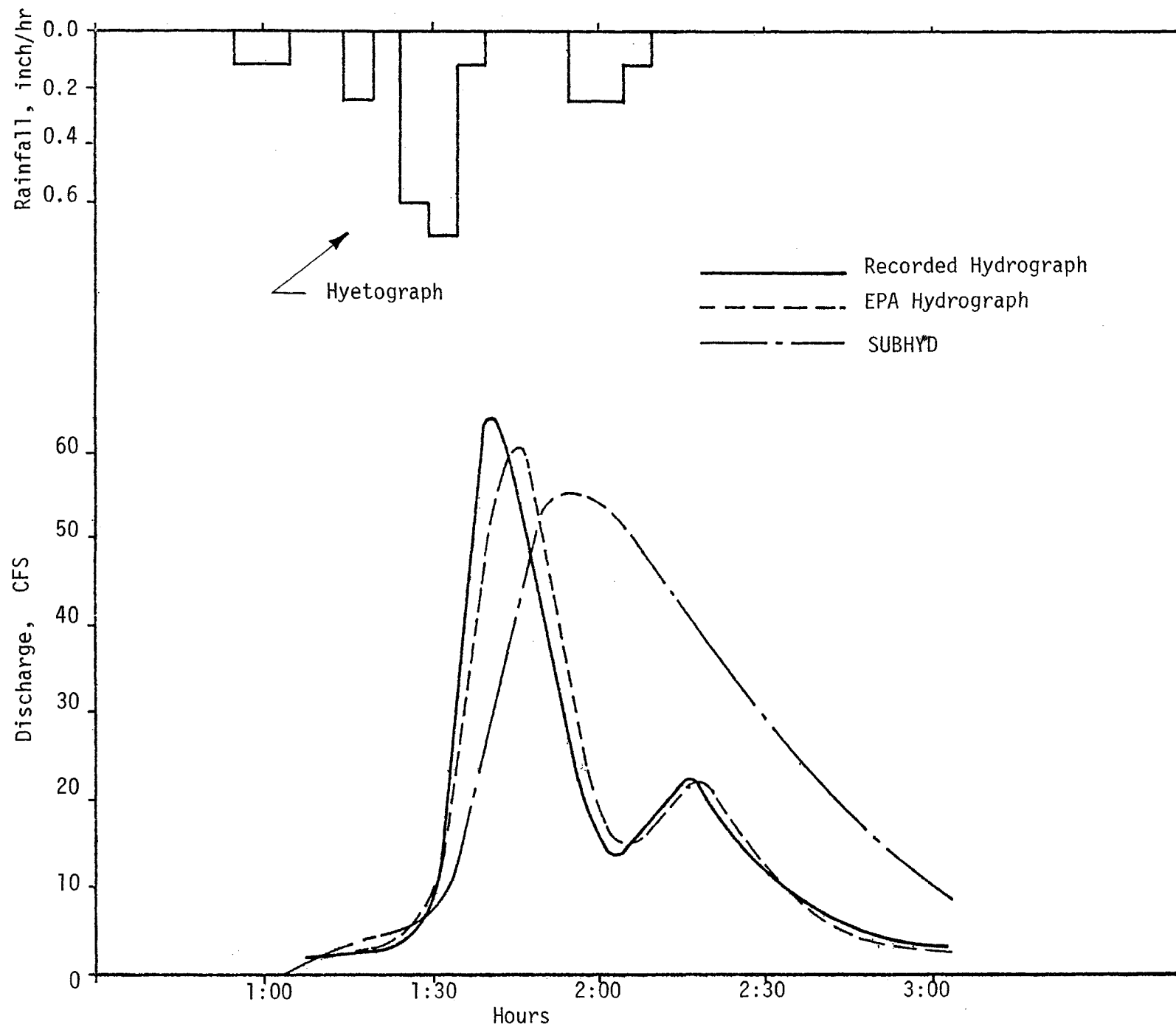


Figure 39 Results from Toronto, Canada — September 1, 1976 (104)

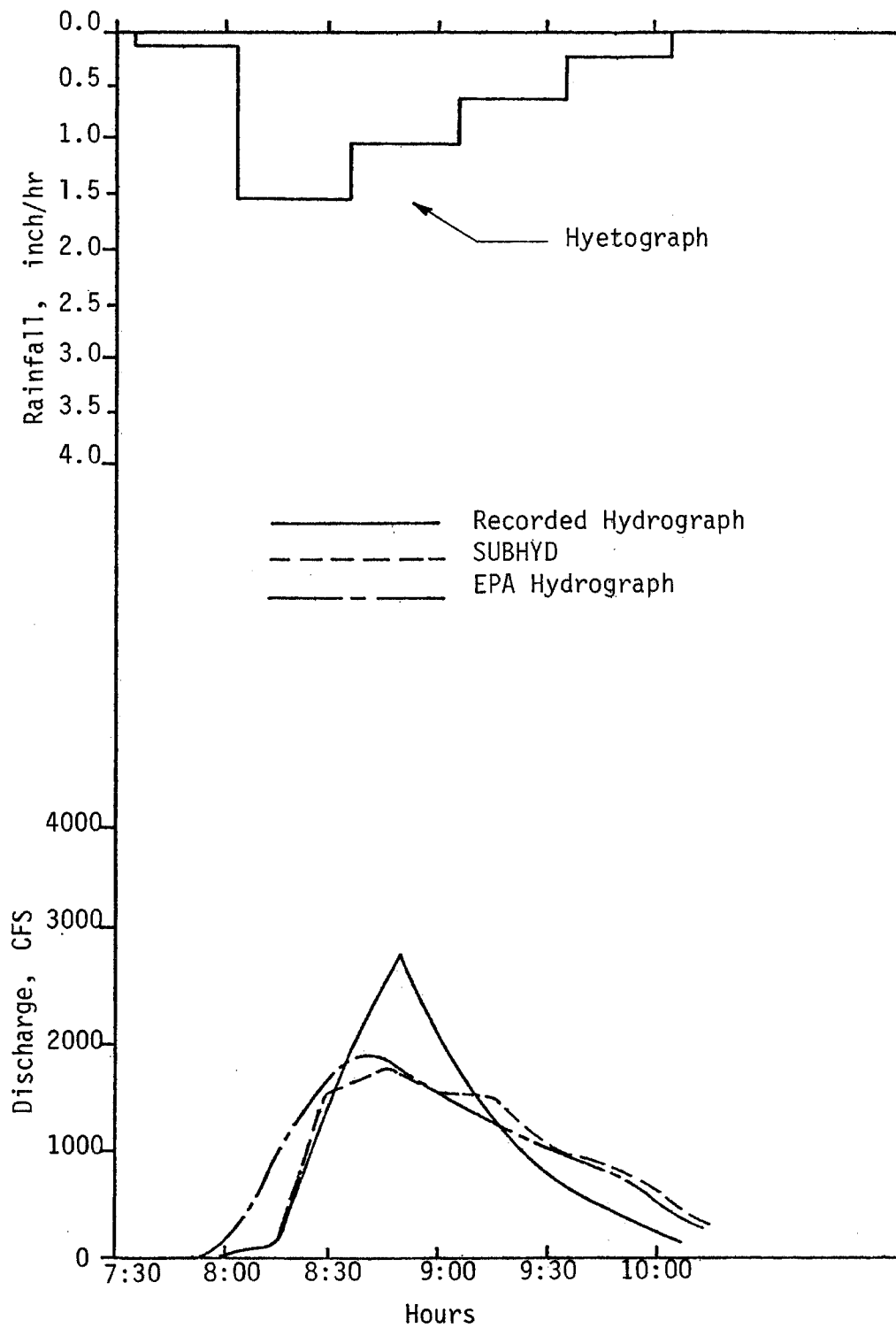


Figure 40 Results from Cincinnati, Ohio - May 12, 1970 (35)

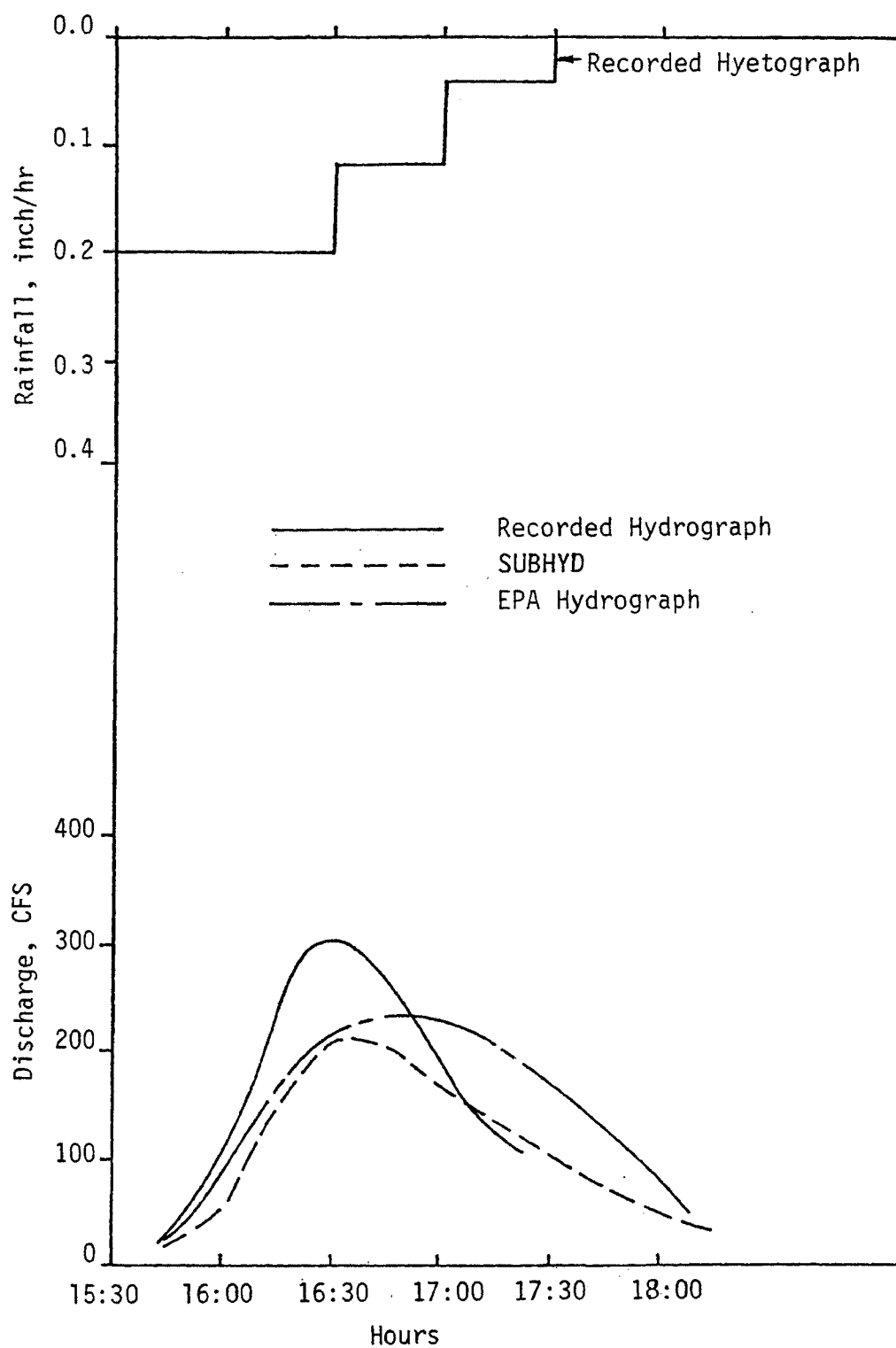


Figure 41 Results from Cincinnati, Ohio - April 1, 1970



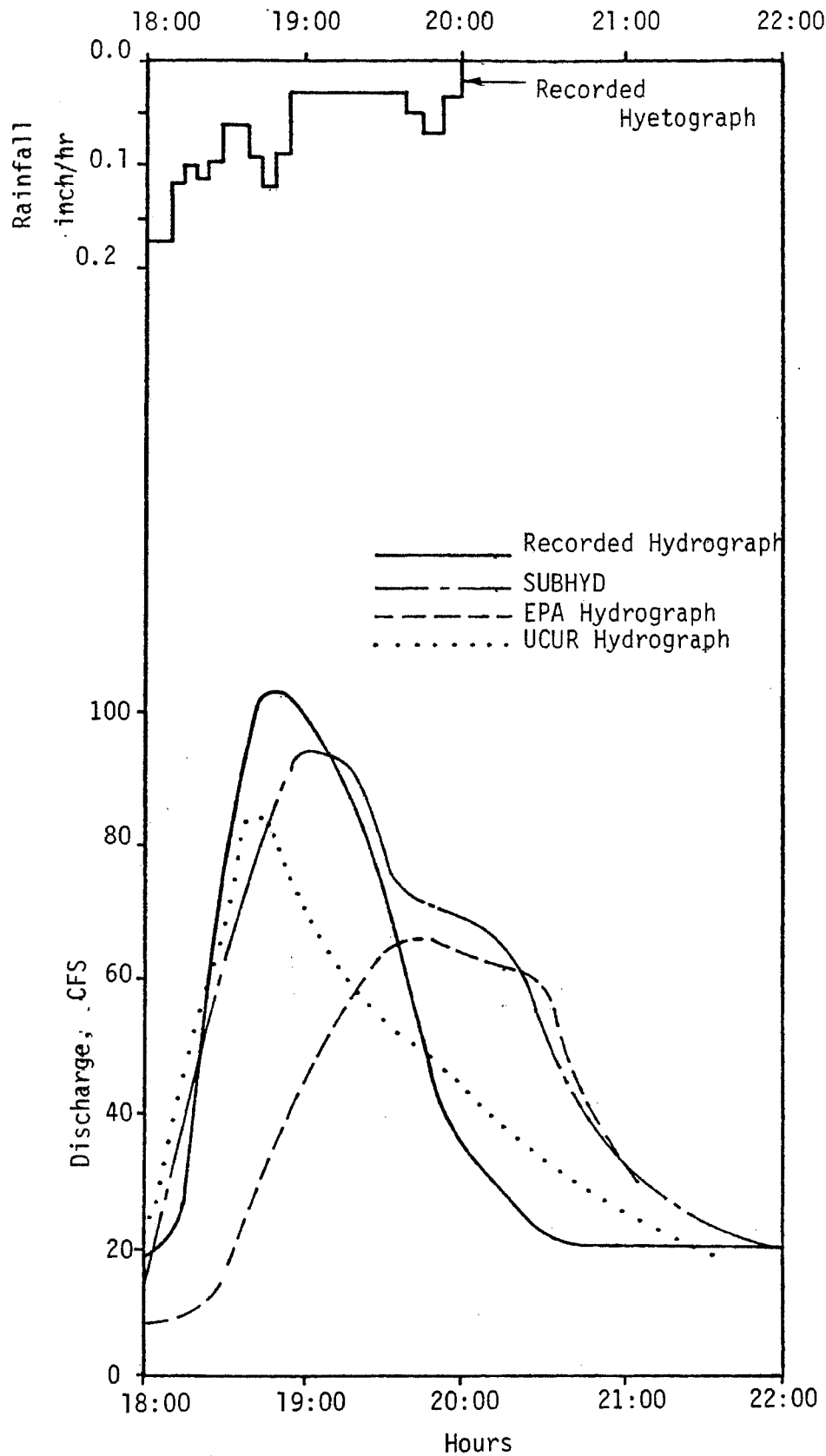


Figure 42 Results from Cincinnati, Ohio -  
November 9, 1970 (110)

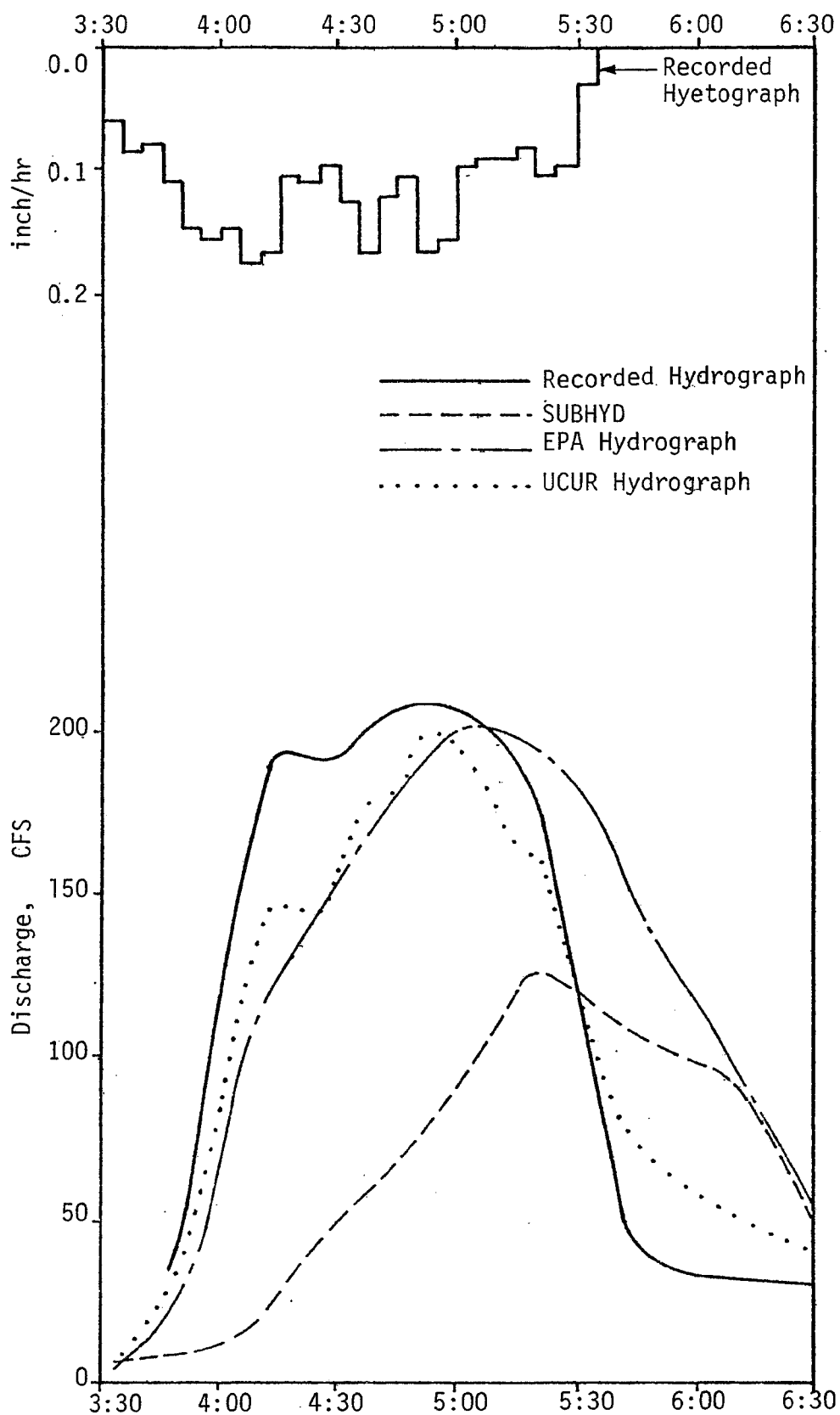


Figure 43 Results from Cincinnati, Ohio -  
May 13, 1971 (110)

## NOTATION

A	- area of a subbasin (acres)
C	- runoff coefficient (dimensionless)
D	- rainfall depth (inches)
d	- minimum diameter or size for a particular conduit (ft)
I	- imperviousness (percent)
L	- overland length (feet)
M	- number of rainfall increments evaluated during total storm duration
MAXDIA	- storage parameter related to maximum conduit size
MAXTIM	- storage parameter related to the maximum time limit allowed for the generation of outfall hydrographs
N	- Manning's roughness coefficient
NN	- number of subbasins
P	- population density in an urbanized area (people/acre)
Q	- discharge for a given conduit or subbasin ( $\text{ft}^3/\text{sec}$ )
$q_p$	- peak discharge from a subbasin or conduit ( $\text{ft}^3/\text{sec}$ )
R	- hydraulic radius (ft)
S	- slope of a subbasin or a pipe (ft/100 ft)
$S_s$	- storage parameter for memory allocation
t	- time from start of storm event
$t_b$	- time base of the total runoff or pipe hydrograph (minutes)
$t_c$	- time of concentration (minutes)
$t_o$	- overland flow time (minutes)
$t_r$	- duration of rainfall increments (minutes)
$t_s$	- time of travel in sewer pipe (minutes)
V	- volume of runoff from a subbasin ( $\text{ft}^3$ )
$V_{su}$	- steady state uniform flow velocity (ft/sec)
x	- fraction of impervious area for a subbasin
$Y_t$	- modified intensity corresponding to the ordinate of the subhydrograph at time t (inches/hr)



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## A P P E N D I X

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*      COMMON/BLOCKC/ N01,N02,N03,N04,N05,N06,N07,N08,N09,N10,N11
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*      DK=0.0
*      CALL INPUT(M,NN,MAXTIM,MAXDIA)
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*      *COMMENT N02=DURAT(M)
*      *COMMENT N03=AREA(NN)
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*      N01=1
*      N02=M+N01
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*      *COMMENT N11=PSLOPE(NN)
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*      *COMMENT N15=NKIND(NN)
*      *COMMENT N16=DISCPT(NN)
*      *COMMENT N17=TIME(M)
*      *COMMENT N18=INTEN(M)
*      *COMMENT N19=RUODEF(NN,M)
*      *COMMENT N20=TIMCON(NN,M)
*      N11=N10+NN
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*COMMENT N54=X(MAXDIA)
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* CALL DATA(S(N01),S(N02),S(N03),S(N04),S(N05),S(N06),S(N07),
* S(N08),S(N09),S(N10),S(N11),S(N12),S(N13),S(N14),S(N15),S(N16),
* M,NN,S(N27),S(N28),S(N29),S(N31),S(N22))
* CALL HYET(S(N01),S(N02),S(N17),S(N18),M,NN)
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* 100 CALL INITIAL(S(N24),S(N25),S(N26),M,NN,S(N30),OK,MAXTIM)
* CALL KINWAV(S(N02),S(N04),S(N05),S(N06),S(N09),S(N12),
* S(N17),S(N18),S(N19),S(N20),S(N21),M,NN)
* CALL SUBHY(S(N02),S(N03),S(N18),S(N19),S(N20),S(N21),
* S(N24),S(N25),S(N26),M,NN,MAXTIM)
* CALL OUTPUT(S(N04),S(N05),S(N06),S(N09),S(N12),S(N22),
* S(N10),S(N11),S(N13),S(N14),S(N15),S(N08),S(N03),S(N17),
* S(N21),S(N23),S(N24),S(N25),S(N26),M,NN,S(N30),MAXTIM,MAXDIA,
* S(N44),S(N45),S(N46),S(N47),S(N48),S(N49),S(N50),S(N51),
* S(N52),S(N53),S(N54),S(N55),S(N56),S(N57),S(N58),S(N59),
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*      CALL ROUTE(S(N04), S(N05), S(N06), S(N09), S(N12), S(N22), *
*      & S(N10), S(N11), S(N13), S(N14), S(N15), S(N08), S(N03), S(N17), *
*      & S(N21), S(N23), S(N24), S(N25), S(N26), M, NN, S(N27), S(N28), S(N29), *
*      & S(N30), S(N32), S(N33), S(N34), S(N35), S(N36), S(N37), S(N38), *
*      & S(N39), S(N40), S(N41), S(N31), S(N42), S(N43), MAXT[M], MAXD[A], *
*      & S(N44), S(N45), S(N46), S(N47), S(N48), S(N49), S(N50), S(N51), *
*      & S(N52), S(N53), S(N54), S(N55), S(N56), S(N57), S(N58), S(N59), S(N60)) *
*****                                B    F
                                  B    F
                                  B    F
                                  B    F
*****                                B    F
DO 105 I=1,NN                       *
*****                                B    F
                                  B    F
                                  B    F
                                  B    F
                                  B    F
*****                                B    F
*      IF(S(N30+I-1).EQ.0.0) GO TO 110 *.....D
*****                                B    F
                                  B    F
                                  B    F
                                  B    F
                                  B    F
*****                                B    F
A .....*      105 CONTINUE          *
*****                                B    F
                                  B    F
                                  B    F
                                  B    F
                                  B    F
*****                                B    F
*      OK=1.0                      *
*****                                B    F
                                  B    F
                                  B    F

```

	I	B	F	J
	0<.....	B	F	0
	I	B	F	
*****		B	F	
* 110 GO TO 100		0	F	
*****			F	
			F	
	0<.....		0	
	I			
*****				
* 115 STOP				
*****				
*****				
* END				
*****				

```

      (ENTRANCE)
      I
*****
* SUBROUTINE INPUTIM,NN,MAXTIM,MAXDIA)
* REAL S,NNLOW,NNHIGH,LTLAND,MANN,LENGTH
* REAL INTER,MAN,NANING,MAX,MANING,LONG
* INTEGER ALIMIT,AINTER
* COMMON/BLOCKA/ DELTA,NE,RATIO,NOPT,DUR,KIND
* COMMON/BLOCKB/ AINTER,NCHECK,NFACT,NCHEX,ARUC,N,NINTER,KAB
* DIMENSION TITLE(15)
*
* NN          IS THE NUMBER OF BASINS OR PIPES
* MAXTIM      IS THE MAXIMUM TIME ALLOWED FOR THE ALLOCATION
*             WITHIN THE PROGRAM
* MAXDIA      IS THE MAXIMUM DIAMETER TO BE DESIGNED FOR IN THE
*             ALLOCATION OF THE PROGRAM
* USE THE EQUATION GIVEN BELOW TO SET UP THE S ARRAY
* S=MAXTIM*(4*M)+(3*(NN*MAXTIM))+(3*(M*NN))+(135*NN)+(14*MAXDIA)
* TITLE       IS AN ALPHA NUMERIC 60 CHARACTERS LONG OF YOUR CHOICE
*
* READ(5,100) TITLE
* 100 FORMAT(10X,15A4)
*
* M           IS THE INTERVALS OF RAINFALL
* DUR         IS THE DURATION OF EQUAL INTERVALS OR IF NOT EQUAL
*             INTERVALS THE LENGTH OF THE VARIABLE IS ZERO
* RATIO       IS THE RATIO OF THE TIME TO PEAK TO THAT OF THE
*             ENTIRE STORM
* DELTA       IS THE INTERVAL OF THE SUBHYDROGRAPH METHOD USUALLY
*             ONE MINUTE
* NOPT        IS THE CONTROL CARD,IF ZERO THE ACTUAL RAINFALL DATA
*             IS USED,IF> ZERO DESIGN STORM DATA IS USED
*
* READ(5,105) M,DUR,RATIO,DELTA,NOPT,MAXTIM
* 105 FORMAT(16X,14,(3X,F7.4),6X,14,5X,15)
*
* AINTER      IS THE DISCHARGE PRINT INTERVAL IN MINUTES AND MUST
*             BE A MULTIPLE OF DELTA
* NFACT       IS THE CONTROL,IF THE TIME OF SEWER IS ADDED TO THE
*             TIME BASE THE VALUE IS ZERO, YOU WILL GET A SYSTEM
*             TIME BASE IF THE VALUE IS ZERO YOU WILL GET A SYSTEM
*             CUTFALL HYDROGRAPH, IF THE VALUE IS >1 EACH INLET AND
*             CONTRIBUTING BASIN WILL BE GIVEN FOR SEPARATE PIPE
*             HYDROGRAPHS
* NCHECK      IS THE CONTROL FOR PRINTING THE HEADING, IF ZERO GENERA
*             SYSTEMS WILL BE GIVEN,IF NOT ZERO DETAILED SYSTEMS
*             WILL BE GIVEN
* ARUC        IS THE PLACEHOLDER FOR THE MAXIMUM DURATION OF STORM
*             USUALLY 150 MIN. , THIS WILL INSURE FULL SATURATION OF
*             THE GROUND SURFACE
* NCHEX       IS ZERO IF ARUC IS ASSUMED AS THE MAXIMUM STORM
*             DURATION WITH A WEIGHTED RUNOFF COEFFICIENT WHILE
*             THE PROGRAM CALCULATES USING KINEMATIC WAVE THEORY
*             IF THE VALUE IS ONE THERE IS ONLY ONE RUNOFF
*             COEFFICIENT FOR THE ENTIRE BASIN

```



```
*      G 'YOUR MAXIMUM TIME LIMIT ALLOWED IS '15,' MINUTES',//,40X,      *
*      G 'YOUR MAXIMUM DIAMETER ALLOWED IS   '15,' INCHES',//,40X,      *
*      G 'YOUR NUMBER OF BASINS AND PIPES IS '13)                        *
```

I  
I  
I

```
*****
*      RETURN
*****
```

```
*****
*      END
*****
```

```

                                (ENTRANCE)
                                I
                                I
*****
*      SUBROUTINE DATAI DEPTH,DURAT,AREA,LTLAND,SLOPE,XFRACT,DISRPT,
*      & NTYPE,MANN,LENGTH,PSLOPE,TIMSEW,NEND,NCOL,NKIND,DSRPT,
*      & M,NN,SHAPE,NNLOW,NNHIGH,PERCEN,DIA)
*      REAL S,NNLOW,NNHIGH,LTLAND,MANN,LENGTH
*      REAL INTEN,MAN,NANING,MAX,MANING,LONG
*      INTEGER ALIMIT,AINTER
*      COMMON/BLOCKA/ DELTA,NE,RATIO,NOPT,DUR,KIND
*      COMMON/BLOCKB/ AINTER,NCHECK,NFACT,NCHEX,AROC,N,NINTER,KAB
*      DIMENSION DEPTH(M),DURAT(M),AREA(M),LTLAND(M),SLOPE(M),
*      & XFRACT(M),DISRPT(M),NTYPE(M),MANN(M),LENGTH(M),PSLOPE(M),
*      & TIMSEW(M),NEND(M),NCOL(M),NKIND(M),DSRPT(M),
*      & SHAPE(M),NNLOW(M),NNHIGH(M),PERCEN(M),DIA(M))
*C
*C      READ RAINFALL DATA
*C
*C      DEPTH      IS THE INCHES OF RAINFALL DURING INTERVAL I
*C      DURAT      IS THE TIME SPREAD OF INTERVAL I, IF DUR>1 THIS VALUE
*C                  IS ZERO, OTHERWISE THIS VALUE IS USED TO FIND THE TIME
*C                  INTERVALS, IF THE INTERVALS ARE EQUAL OR IF THE VALUES
*C                  ARE UNEQUAL THE INTERVALS ARE BYPASSED
*C
*      READ(5,150) (DEPTH(I),DURAT(I), I=1,M)
*****
                                I
                                I
*****
*      IF(DUR.EQ.0.0) GO TO 105
*****
                                I
                                I
                                I
                                I
A-----*      DO 100 I=1,M
A-----*
A-----*
A-----*
A-----*
A-----*      100 DURAT(I)=DUR
A-----*
*****
                                I
                                I<-----0
                                I
*****
*      105 CONTINUE
*C
*C      READ STORM SEWER SYSTEM PARAMLTERS
*C
*C
*C      INCREMENTING TO ZERO
*C

```

	I	B	J	N
	O<.....	B	J	O
	I	B	J	
*****		B	J	
* 125 MAN=MANN(NO)	*	B	J	
* IF(NCHECK.EQ.0) TIMSEW(NO)=LTLAND(NO)/10.0	*	B	J	
*****		B	J	
	I	B	J	
	I	B	J	
	I	B	J	
*****		B	J	
* IF(NO.GE.NE) GO TO 130	*.....O	B	J	
*****		B	F	J
	I	B	F	J
	I	B	F	J
	I	B	F	J
*****		B	F	J
* GO TO 110	*.....>A	B	F	J
*****		B	F	J
		B	F	J
		B	F	J
		B	F	J
		B	F	J
*****		B	F	J
*C	*	B	F	J
*C SPECIAL CASE FOR FIRST TIME THRU DATA DOESN'T HAVE TO BE READ IN	*	B	F	J
*C ANY PARTICULAR ORDER WILL SORT OUT	*	B	F	J
*C	*	B	F	J
*****		B	F	J
	I	B	F	J
	O<.....	B	J	O
	I	B	J	
*****		B	J	
* 130 NE=NO	*	B	J	
*****		B	J	
	I	B	J	
	I	B	J	
	I	B	J	
*****		B	J	
* GO TO 110	*.....>A	B	J	
*****		B	J	
		B	J	
		B	J	
		B	J	
*****		B	J	
*C	*	B	J	
*C PIPE SYSTEM DATA	*	B	J	
*C	*	B	J	
*****		B	J	
	I	B	J	

```

      I      B      J
      I      B      J
      O<.....B.....0
      I      B
*****
* 135 SHAPE(NO)=SIZE      *
*   LENGTH(NO)=ALENGT    *
*   PSLOPE(NO)=VALUE     *
*   NNLOW(NO)=RUUGH      *
*   NNHIGH(NO)=PER       *
*   TIMSEW(NO)=LENGTH(NO)/120.0
*   NEND(NO)=NEND0       *
*   NCOL(NO)=NCOLL       *
*   NKIND(NO)=ATYP       *
*   DSCRPT(NO)=DSPT      *
*   PERCEN(NO)=PERCE     *
*   DIA(NO)=0.0          *
*****
      I      B
      I      B
      I      B
*****
*   GO TO 110             *.....0
*****

*****
* 140 CONTINUE           *
*   IF(NN.NE.NE) WRITE(6,145)
* 145 FORMAT(10X,'YOUR INPUT DATA DOES NOT MATCH ERROR')
* 150 FORMAT(12X,F8.4,10X,F10.4)
* 155 FORMAT(1X,12,2X,F6.2,4X,F7.2,1X,F6.4,2X,F8.4,2X,F6.4,2X,12,1X,12,
*   61X,12,1X,12,A4,8X,F4.2)
*****
      I
      I
*****
*   RETURN               *
*****

*****
*   END                  *
*****

```

```

      (ENTRANCE)
      I
      I
      *****
      * SUBROUTINE HYET(DEPTH, DURAT, TIME, INTEN,M,NN) *
      * REAL S,NNLOW,NNHIGH,LTLAND,MANN,LENGTH *
      * REAL INTEN,MAK,NANING,MAX,MANING,LCNG *
      * INTEGER ALIMIT,AINTER *
      * COMMON/BLOCKA/ DELTA,NE,RATIO,NOPT,DUR,KIND *
      * COMMON/BLOCKB/ AINTER,NCHECK,NFACT,NCHX,AROC,N,NINTER,KAB *
      * DIMENSION DEPTH(M), DURAT(M), TIME(M), INTEN(M) *
      * TIM=0.0 *
      *****
      I
      I
      I
      *****
      * DO 100 KT=1,M *
      *****
      I
      I
      I
      *****
      * TIME(KT)=TIM+DURAT(KT) *
      * TIM=TIME(KT) *
      *****
      I
      I
      I
      *****
      * 100 CONTINUE *
      *****
      I
      I
      *****
      *C *
      *C OPTION CONTROL *
      *C IF ZERO ACTUAL RAINFALL DATA,IF>1 DESIGN STORM DATA *
      *C *
      *****
      I
      I
      I
      *****
      * IF(NOPT.NE.0) GO TO 110 *
      *****
      I
      I

```

```
A ..... * DO 125 KT=1,M *
```

```
*C SEE IF TIME HAS REACHED RECEDED LIMB OF HYDROGRAPH IF NOT  
*C FIND ASCENDING EDGE OF HYDROGRAPH
```

```
*C  
IF(KT.GT.NPEAK) GO TO 120.....O  
  
T2=KT*DURAT(1)  
DT=(T2-T1)/RATIO  
KAT=(NPEAK-KT)+1  
*C FOR FIRST TIME THRU SPECIAL CASE FOR RISING LIMB OF HYDROGRAPH  
*C  
IF(KAT.LE.1) GO TO 115.....F.....  
  
INTEN(KAT)=60.0*(DEPH(KT)/DT)  
T1=T2
```

```

*****
*      GO TO 125
*****
*****
*C      INCREMENT HYDROGRAPH DEPTH TO PRECEDING DEPTH
*C
*****
      I
      OK.....>V
      I
*****
*      115 LT=KAT-1
*C
*C      FIND RISING SIDE OF HYDROGRAPH
*C
*      INTEN(KAT)=60.0*(DEPTH(KT)-DEPTH(LT))/DT
*      T1=T2
*****
      I
      I
      I
*****
*      GO TO 125
*****
      OK.....>V
      I
*****
*      120 KAT=KT-NPEAK
*      T2=KAT*DURAT(1)
*      DT=(T2-T1)/RATIO
*C
*C      FIRST TIME THRU OR RECEDING LIMB OF HYDROGRAPH
*C
*****
      I
      I
      I
*****
*      IF(KAT.NE.M) GO TO 125
*****
      I

```

A		I	F
A		I	F
A		I	F
A	*****		F
A	*      LT=KT-1	*	F
A	*      INTEN(KT)=6G.0*(DEPTH(KT)-DEPTH(LT))/DT	*	F
A	*      T1=T2	*	F
A	*****		F
A		I	F
A		0<.....0	F
A		I	
A	*****		
.....*	125 CONTINUE	*	
	*****		
		I	
		I	
	*****		
	*      RETURN	*	
	*****		
	*****		
	*      END	*	
	*****		



[illegible]

```

*****
*      IF(OK.EQ.1.0) GO TO 115      *.....0
*****                                B
                                      B
                                      B
                                      B
*****                                B
*      DO 110 I=1,NN                *      B
*****                                B
                                      B
                                      B
                                      B
*****                                B
*      GOOD(I)=0.0                  *      B
*****                                B
                                      B
                                      B
                                      B
*****                                B
*      110 CONTINUE                 *      B
*****                                B
                                      B
                                      B
                                      B
*****                                B
*      115 RETURN                    *      B
*****                                B
                                      B
                                      B
                                      B
*****                                B
*      END                          *
*****

```

```
(ENTRANCE)
      I
*****
      SUBROUTINE KINWAV( DURAT, LTLAND, SLOPE, XFRAC, MANN, TIMSEW,
      & TIME, INTEN, RUCOE, TIMCON, TBASE,M,NN)
      REAL S,NNLOW,NNHIGH,LTLAND,MANN,LENGTH
      REAL INTEN,MAN,NANING,MAX,MANING,LONG
      INTEGER ALIMIT,AINTER
      COMMON/BLOCKA/ DELTA,NE,RATIO,NOPT,DUR,KIND
      COMMON/BLOCKB/ AINTER,NCHECK,NFACT,NCHX,AROL,N,NINTER,KAB
      DIMENSION DURAT(M), LTLAND(NN), SLOPE(NN), XFRAC(NN), MANN(NN),
      & TIMSEW(NN), TIME(M), INTEN(M), RUCOE(NN, M), TIMCON(NN, M),
      & TBASE(NN, M)
      *C
      *C   LARGEST VALUE WHEN USING DRAINAGE BASIN DATA MAIN PROGRAM
      *C
*****
      I
      I
*****
DO 125 NPOS=1,NE
*****
      I
      I
*****
DO 125 KT=1,M
*****
      I
      I
*****
      *C
      *C   OPTION CCATROL
      *C
*****
      I
      I
*****
      IF(NCHX.EQ.O) GO TO 100
*****
      I
      I
*****
      *C
      *C   NOT A CONSTANT RUNOFF COEFFICIENT FOR AREAS
      *C
      *C   RUCOE(INPOS,KT)=XFRAC(INPOS)*(TIME(KT)/(TIME(KT)+8.0))+11.0-
      *C   (XFRAC(INPCS))*10.5*(TIME(KT)/(TIME(KT)+15.0))
*****
      I
```

```
A B      I      B
A B      I      B
A B      I      B
A B      I      B
A B      * .....0
A B      * GO TO 105                                * .....0
A B      * .....F
A B      O<.....D
A B      I      F
A B      I      F
A B      * .....F
A B      * 100 ROCOEF(NPOS,KT)=(XFRAC(TNPOS)*(AROC/(AROC+8.)))+ * .....F
A B      * C(((1.-XFRAC(TNPOS))*0.5*AROC/(AROC+15.0))) * .....F
A B      * .....F
A B      I      F
A B      O<.....0
A B      I      I
A U      * .....0
A B      * 105 IF(INTEN(KT).EQ.0.0) GO TO 120          * .....0
A B      * .....B
A B      I      B
A B      I      B
A B      I      B
A B      * .....B
A B      * IF(SLOPE(NPOS).EQ.0.0) GO TO 115          * .....0
A H      * .....B
A U      * .....F
A B      I      B
A B      I      B
A B      I      B
A H      * .....B
A B      * TIMCON(NPOS,KT)=0.9*(ILTAND(NPOS)**0.6)*(MANN(NPOS)**0.6)/ * .....B
A B      * C(((INTEN(KT)*ROCOEF(NPOS,KT))**0.4)*(SLOPE(NPOS)**0.3)) * .....B
A B      *C * .....F
A B      *C OPTION CONTROL * .....B
A B      *C INLET AND ASSOCIATED BASIN CONTRIBUTE TO SEPARATE PIPE * .....B
A B      *C HYDROGRAPHS * .....B
A B      *C GENERAL ALL BASINS LUMPED TOGETHER * .....B
A B      *C * .....B
A B      * IF(INFACT.NE.O) TIMCCN(NPOS,KT)=TIMCON(NPOS,KT)+TIMSEW(NPOS) * .....B
A B      * VALUE=TIMCCN(NPOS,KT)/DELTA * .....B
A B      * NVAL=VALUE * .....B
A U      * .....B
A B      I      B
A B      I      B
A B      I      B
A B      I      B
A B      * .....H
A B      * IF(VALUE.EQ.NVAL) GO TO 110                * .....U
A B      * .....F
A B      I      B
```

```
A B      I
A B      I
A B      I
A B      I
A B      *C
A B      *C TRUNCATION UP TO MAKE NEXT INTERVALS
A B      *C
A B      * NVAL=NVAL+I
A B      * TIMCON(NPOS,KT)=NVAL*DELTA
A B      *****
A B      I
A B      OK<.....F.....0
A B      I
A B      *****
A B      * 110 CCNTINUE
A B      * TBASE(NPOS,KT)=TIMCONINPOS,KT)+DURAT(KT)
A B      *****
A B      I
A B      I
A B      I
A B      *****
A B      * GO TO 125
A B      *****
A B      OK<.....F.....0
A B      I
A B      I
A B      *****
A B      * 115 TIMCON(NPOS,KT)=0.0
A B      *****
A B      I
A B      OK<.....0
A B      I
A B      *****
A B      * 120 TBASE(NPOS,KT)= 0.0+DURAT(KT)
A B      *****
A B      I
A B      OK<.....0
A B      I
A B      *****
A B      * 125 CONTINUE
A B      *****
A B      I
A B      I
A B      *****
A B      * RETURN
A B      *****
```



```

      (ENTRANCE)
      I
      I
      *****
      * SUBROUTINE SUBHY( DURAT, AREA, INTEN, ROCCOF, TIMCON, TBASE,
      * & ATIMES, TIMES, FLOW, M, NN, MAXTIM)
      * REAL S, NNLOW, NNHIGH, LTLAND, MANN, LENGTH
      * REAL INTEN, MAN, NANNING, MAX, MANING, LONG
      * INTEGER ALIMIT, AINTER
      * COMMON/BLCKA/ DELTA, NE, RATIO, NUPT, DUR, KIND
      * COMMON/BLCKB/ AINTER, NCHECK, NFACT, NCHEX, ARUC, N, NINTER, KAB
      * DIMENSION DURAT(M), AREA(NN), INTEN(M), ROCCOF(NN, M),
      * & TIMCON(NN, M), TBASE(NN, M)
      * DIMENSION ATIMES(MAXTIM), TIMES(NN, MAXTIM), FLOW(NN, MAXTIM)
      * MAXTI=MAXTIM-5
      *****
      I
      I
      *****
      * DO 165 N=1, NE
      *****
      I
      I
      *****
      * KAB=0
      * MOVE=0
      * NMOVE=0
      *****
      I
      I
      *****
      * DO 165 KT=1, M
      *****
      I
      I
      *****
      * QP=0.
      * TIM=DELTA
      * KNT=1
      *****
      I
      I
      *****
      * IF( INTEN(KT).EQ.0.0.OR. AREA(N).EQ.0.0) GO TO 140
      *****
      I

```

[illegible]



	I	B	D	F	H	J	N
*****	I	B	O	F	H	J	N
* GO TO 135		B	O	F	H	J	N
*****		B	O	F	H	J	N
*****		B	O	F	H	J	N
*C		B	O	F	H	J	N
*C CASE B FOR T<TC		B	O	F	H	J	N
*C		B	O	F	H	J	N
*****		B	O	F	H	J	N
	I	B	O	F	H	J	N
	O<.....	B	O	F	H	J	N
*****	I	B	O	F	H	J	N
* 120 SUBHYD=(INTEN(KT)*TIM/TIMCON(N,KT))*(1.0/60.0)		B	O	F	H	J	N
* QP=ROCOEF(N,KT)*INTEN(KT)*AREA(N)		B	O	F	H	J	N
*****		B	O	F	H	J	N
	I	B	O	F	H	J	N
	I	B	O	F	H	J	N
*****		B	O	F	H	J	N
* GO TO 135		B	O	F	H	J	N
*****		B	O	F	H	J	N
*C		B	O	F	H	J	N
*C CASE B FOR TC<T<TR		B	O	F	H	J	N
*C		B	O	F	H	J	N
*****		B	O	F	H	J	N
	I	B	O	F	H	J	N
	O<.....	B	O	F	H	J	N
*****	I	B	O	F	H	J	N
* 125 SUBHYD=INTEN(KT)*(1.0/60.0)		B	O	F	H	J	N
* QP=ROCOEF(N,KT)*INTEN(KT)*AREA(N)		B	O	F	H	J	N
*****		B	O	F	H	J	N
	I	B	O	F	H	J	N
	I	B	O	F	H	J	N
*****		B	O	F	H	J	N
* GO TO 135		B	O	F	H	J	N
*****		B	O	F	H	J	N
	I	B	O	F	H	J	N

[illegible]

```

A B      O<.....J N
A B      I.....D.....O
A B      * .....J
A B      *****  

A B      * 155 MOVE=(DURAT(ILT)/DELTA)+MOVE  

A B      *C  

A B      *C BEGINNING OF STORM  

A B      *C  

A B      * KAB=KNT+MCVE  

A B      *****  

A B      I  

A B      I  

A B      I  

A B      *****  

A B      * IF(INTEN(KT).EQ.0.0.OR.AREA(N).EQ.0.0) GO TO 160  

A B      * .....>V J  

A B      *****  

A B      I  

A B      I  

A B      I  

A B      *****  

A B      * COUNT=KNT*DELTA  

A B      *C  

A B      *C DEVELOPMENT OF SUBHYDROGRAPHS INTO TOTAL HYDROGRAPH  

A B      *C  

A B      * IF(KAB.GE.MAXTI) COUNT=TBASE(N,KT)  

A B      * FLOW(N,KAB)=FLOW(N,KAB)+SUBHYD*ROCOEF(N,KT)*AREA(N)*(1.0/12.0)*  

A B      * E43560.0*(1.0/60.0)  

A B      *****  

A B      I  

A B      I  

A B      I  

A B      *****  

A B      * GO TO 150  

A B      * .....B...O J  

A B      *****  

A B      B J  

A B      O<.....O J  

A B      I J  

A B      *****  

A B      * 160 CONTINUE  

A B      *C  

A B      *C PAST MOVEMENT OF HYDROGRAPH TO SET UP NEW ORIGIN  

A B      *C  

A B      *****  

A B      I  

A B      O<.....J  

A B      I O  

A B      *****  

A B      * 165 NMOVE=MOVE  

A B      *****  

A B      I

```

I  
I  
I  
I

\*\*\*\*\*  
\* RETURN \*  
\*\*\*\*\*

\*\*\*\*\*  
\* END \*  
\*\*\*\*\*

```

      I
      |
*****
* SUBROUTINE OUTPUT( LTLAND, SLOPE, XFRAC, MANN, TIMSEW, DIA,
*   & LENGTH, PSLOPE, NEND, NCOL, NKIND, NYTYPE, AREA, TIME, TBASE,
*   & ALIMIT, ATIMES, TIMES, FLOW, H, NN, GOOD, MAXTIM, MAXDIA,
*   & ZEDA, ANGLE, WETPER, AREAF, MANING, LCNG, SS, PN, DTOD, Y, X, DEEP, HYDRAD,
*   & VELOCITY, TIMSER, TOTTS, AVETIM, PERCEN, R1, R2, R3, R4, R5, SHAPE)
* REAL S, NNLOW, NNHIGH, LTLAND, MANN, LENGTH
* REAL INTEN, MAN, NANING, MAX, MANING, LONG
* INTEGER ALIMIT, AINTER
* COMMON/BLOCKA/ DELTA, NE, RATID, NOPT, DUR, KIND
* COMMON/BLOCKB/ AINTER, NCHECK, NFACT, NCHEX, AROC, N, NINTER, KAB
* DIMENSION LTLAND(NN), SLOPE(NN), XFRAC(NN), MANN(NN),
*   & TIMSEW(NN), DIA(NN), LENGTH(NN), PSLOPE(NN), NEND(NN), NCOL(NN),
*   & NKIND(NN), NYTYPE(NN), AREAINN), TIMEIN), TBASE(INN, MI),
*   & ALIMIT(NN), GOOD(NN)
* DIMENSION ATIMES(MAXTIM), TIMES(INN, MAXTIM), FLOW(INN, MAXTIM)
* DIMENSION ZEDA(MAXDIA), ANGLE(MAXDIA), WETPER(MAXDIA), SHAPE(INN),
*   & AREAF(MAXDIA), MANING(MAXDIA), LCNG(INN), SS(MAXDIA),
*   & PN(MAXDIA), DTOD(MAXDIA), Y(MAXDIA), X(MAXDIA), DEEP(MAXDIA),
*   & HYDRAD(MAXDIA), VELOCY(MAXDIA), TIMSER(MAXDIA), TOTTS(INN),
*   & AVETIM(INN), , PERCEN(NN), R1(NN), R2(NN), R3(NN), R4(NN), R5(NN)
* MAXTI=MAXTIM-5
*
* C
* C CALCULATE ALIMIT
* C FINDS THE LONGEST TIME FOR BEGINNING OF RAINFALL TO END OF RUNOFF
* C
*****
      I
      |
*****
DO 115 N=1, NE
*****
      I
      |
*****
* MAX=0
*****
      I
      |
*****
DO 110 KT=1, M
*****

```

```

      I
      I
      I
      I
*****
*      LT=KT-1
*****
      I
      I
      I
*****
*      IF(KT.EQ.1) GO TO 100
*****
      I
      I
*****
*      MIN=TIME(LT)+TBASE(N,KT)
*****
      I
      I
      I
*****
*      GO TO 105
*****
*****
*C      PROPER SELECTION FOR LARGEST TIME VALUE FROM BEGINNING OF STORM
*C
*****
      I
      OK.....0
      I
*****
*      100 MIN=TBASE(N,KT)
*****
      I
      OK.....0
      I
*****
*      105 IF(MAX.GE.MIN) GO TO 110
*****
      I
      I
      I
*****
*      MAX=MIN
*****
      I
      I

```

[illegible]

```

*****
*      IF(NCHECK.EQ.0) GO TO 130
*****
*      IF(GOOD(N).EQ.0.0) GO TO 125
*****
*      120 IF(NTYPE(N).EQ.1) WRITE(6,240) N,AREA(N),MANN(N),LTAND(N),
*           & SLOPE(N),XFRAC(N)
*****
*      125 GO TO 135
*****
*      130 IF(GOOD(N).EQ.0.0) GO TO 135
*****
*      IF(NTYPE(N).EQ.1) WRITE(6,255) N,AREA(N),MANN(N),LTAND(N),
*           & SLOPE(N),XFRAC(N)
*      C
*      C      NUMBER OF TIME INTERVALS TO BE PRINTED
*      C
*****
*      135 NO=ALIMIT(N)/DELTA
*           KAC=1
*****

```



```
A A A A A I  
A B ..... * DO 145 KAB=1,NO *  
A B I  
A B I  
A B  
A B *****  
A B * TIMES(N,KAB)=KAB*DELTA *  
A B * IF(KAB.GE.MAXT) KAB=NO *  
A B *****  
A B I  
A B I  
A B  
A B *****  
A B * IF(KAB.GE.MAXT) GO TO 145 *.....0  
A B *****  
A B I  
A B I  
A B  
A B *****  
A B * IF(GOOD(N).EQ.0.0) GO TO 140 *.....8  
A B *****  
A B I  
A B I  
A B  
A B *****  
A B * IF(FLOWIN,KAB).EQ.0.0) GO TO 140 *.....>V  
A B *****  
A B I  
A B I  
A B  
A B *****  
A B * IF(NTYPE(N).EQ.1.AND.KAC.EQ.NINTER) WRITE(6,245) TIMES(N,KAB), *  
A B * C FLOWIN,KAB) *  
A B *****  
A B I  
A B O<.....0  
A B I  
A B  
A B *****  
A B * 140 KAC=KAC+1 *  
A B * IF(KAC.EQ.NKAC)KAC=1 *  
A B *****  
A B I  
A B O<.....0  
A B I
```

```

A B
A B
A B
.....* 145 CONTINUE
*****
I
I
I
*****
*C      OPTION CONTROL
*****
I
I
I
*****
*      IF(INCHECK.EQ.0) GO TO 270
*****
I
I
I
*****
*C
*C      CALCULATE PIPE HYDROGRAPHS
*C
*      TLIMIT=0.0
*      LPOS=1
*      CLIMIT=0.0
*      NPOS=1
*****
I
I
I
*****
*      GO TO 175
*****
O<.....D
I      B      F
I      B      F
I      B      F      J
*****
+ 150 IF(ENEND(NPOS).EQ.NPOS) GO TO 165
*****
I
I
I
*****
*      IF(ALIMIT(NPOS).GT.TLIMIT) GO TO 155
*****
O.....D
I      B      D      F      J      N
I      B      D      F      J      N
I      B      D      F      J      N

```

[illegible]

```

*****
*      IF(KAC.EQ.NINTER) GO TO 205
*****
*****
*      GO TO 210
*****
*****
*      OK.....
*****
*      205 KAC=0
*      IF(INKIND(NPOS).EQ.1) WRITE(6,245) ATIMES(KAB),FLOW(NPOS,KAB)
*****
*****
*      OK.....
*****
*      210 IF(INCOL(LPOS).EQ.NPOS) GO TO 215
*****
*****
*      GO TO 220
*****
*****
*      OK.....
*****
*      215 TLIMIT=CLIMIT
*      CLIMIT=0.
*****
*****
*      OK.....
*****
*****
*      220 IF(TIMES(NPOS,KAB).GE.ALIMIT(NPOS).AND.TIMES(NPOS,KAB)
*      & .GE.TLIMIT) GO TO 150
*****
*****

```

```

*****
*      KAB=KAB+1
*      KAC=KAC+1
*****
I
I
I
*****
*      GO TO 185
*****
OK.....
I
I
I
*****
*      225 IF(NPOS.EC.NE) GO TO 270
*****
I
I
I
*****
*      IF(NPOS.EQ.NEND(NPOS)) GO TO 235
*****
I
I
I
*****
*      WRITE(6,230)
*      230 FORMAT(10X,'YOUR GEOMETRY DOES NOT CHECK ERROR',/)
*****
I
OK.....
I
*****
*      235 LPOS=NPOS
*      NPOS=NPOS+1
*      TLIMIT=BLIMIT
*****
I
I
I
*****
*      GO TO 175
*****
*****
*      240 FORMAT(1H1,13X,'INLET HYDROGRAPH, INLET NO. ',13,/,5X,'DRAINAGE',*
*      G' AREA = ',F5.1,'ACRES, MANNINGS N = ',F5.3,/, OVERLAND LENGTH =
*      G',F5.0,/, FT,/,5X,'SLOPE = ',F7.3,/'FT/FT, FRACTION OF IMPERVIOUS
*      G', 'SURFACE = ',F7.2,/,12X,'TIME',14X,'INLET DISCHARGE'
*      G',/,9X,'(MINUTES)',17X,'(CFS)')
*      245 FORMAT(11X,F6.1,16X,F8.2)
*      250 FORMAT(1H1,13X,'THIS IS A CIRCULAR PIPE WITH THE',/,

```

```

* 255 FORMAT(1H1,13X,'SYSTEM HYDROGRAPH, INLET NO. ',I3,/,5X,'DRAINAGE' *
* 6,' AREA = ',F5.1,' ACRES, MANNINGS N = ',F5.3,' OVERLAND LENGTH = ' *
* 6,'F5.0,' FT.,/,5X,'SLOPE = ',F7.3,' FT/FT, FRACTION OF IMPERVIOUS *
* 6,' SURFACE = ',F7.2,/,12X,'TIME',14X,'SYSTEM DISCHARGE' *
* 6,/,8X,'(MINUTES)',17X,'(CFS)') *
* 260 FORMAT(1H1,13X,'THIS IS A SEMI-ELLIPTICAL PIPE WITH',/, *
* 613X,'R1 = ',F10.2,' INCHES',5X,'R2 = ',F10.2,' INCHES',5X,'R3 = ', *
* 6F10.2,' INCHES',/,26X,'R4 = ',F10.2,' INCHES',5X,'R5 = ',F10.2, *
* 6' INCHES',/, *
* 613X,'PIPE HYDROGRAPH, PIPE NO. ',I3,/,5X,'PIPE LENGTH= ', *
* 6F6.1,' FT, PIPE DIA. = ',F6.2,' INCHES, SLOPE = ',F5.1, *
* 6' PERCENT',/,5X,'ASSUMED TRAVEL TIME IN PIPE IS ',F6.2,' MINUTES', *
* 6///,12X,'TIME',14X,'PIPE DISCHARGE',/,9X,'(MINUTES)',17X,'(CFS)') *
* 265 FORMAT(1H1,13X,'THIS IS A RECTANGULAR TRUNK SEWER WITH THE',/, *
* 613X,'HEIGHT = ',F10.2,' INCHES',5X,'AND THE LENGTH = ',F10.2, *
* 6' INCHES',/, *
* 613X,'PIPE HYDROGRAPH, PIPE NO. ',I3,/,5X,'PIPE LENGTH= ', *
* 6' PERCENT',/,5X,'ASSUMED TRAVEL TIME IN PIPE IS ',F6.2,' MINUTES', *
* 6///,12X,'TIME',14X,'PIPE DISCHARGE',/,9X,'(MINUTES)',17X,'(CFS)') *
*****
1
0<.....0
1
*****
* 270 RETURN *
*****

*****
* END *
*****

```

(ENTRANCE)

I  
I

```
*****
* SUBROUTINE ROUTE1 LTLAND, SLOPE, XFRAC, MANN, TIMSEW, DIA,
* & LENGTH, PSLOPE, NEND, NCOL, NKIND, NTYPE, AREA, TIME, TBASE,
* & ALIMIT, ATIMES, TIMES, FLOW, M, NN, SHAPE, NNLOW, NNHIGH, GOOD,
* & AXFLOW, NANING, MAX, DIA, MM, R1, R2, R3, R4, R5, PERCEN, EPSILN, OLDDIA,
* & MAXTIM, MAXDIA,
* & ZEDA, ANGLE, WETPER, AREAFL, MANING, LCNG, SS, PN, OTOD, Y, X, DEEP, HYDRAD,
* & F6.1, ' FT, SLOPE = ', F5.1,
* & VELOCITY, TIMSER, TOTTS, AVETIM)
* REAL S, NNLOW, NNHIGH, LTLAND, MANN, LENGTH
* REAL INTE, MAN, NANING, MAX, MANING, LONG
* INTEGER ALIMIT, AINTER
* COMMON/BLOCKA/ DFLTA, NE, RATIO, NOPT, DUR, KIND
* COMMON/BLOCKB/ AINTER, NCHECK, NFACT, NCHEX, AROC, N, NINTER, KAB
* DIMENSION ATIMES(MAXTIM), TIMES(INN, MAXTIM), FLOW(INN, MAXTIM)
* DIMENSION ZEDA(MAXDIA), ANGLE(MAXDIA), WETPER(MAXDIA),
* & AREAFL(MAXDIA), MANING(MAXDIA), LONG(INN), SS(MAXDIA),
* & PN(MAXDIA), OTOD(MAXDIA), Y(MAXDIA), X(MAXDIA), DEEP(MAXDIA),
* & HYDRAD(MAXDIA), VELOCITY(MAXDIA), TIMSER(MAXDIA), TOTTS(INN),
* & AVETIM(INN)
* DIMENSION AXFLOW(INN), MAX(INN, MAXTIM), DIA(INN), MM(INN), R1(INN), R2(INN),
* & R3(INN), R4(INN), R5(INN), NANING(INN), PERCEN(INN), EPSILN(INN)
* DIMENSION LTLAND(INN), SLOPE(INN), XFRAC(INN), MANN(INN),
* & TIMSEW(INN), DIA(INN), LENGTH(INN), PSLOPE(INN), NEND(INN), NCOL(INN),
* & NKIND(INN), NTYPE(INN), AREA(INN), TIME(INN), TBASE(INN, M),
* & ALIMIT(INN), SHAPE(INN), NNLOW(INN), NNHIGH(INN), GOOD(INN), OLDDIA(INN)
* MAXTI=MAXTIM-5
*****
```

```
*C
*C FINDING THE MAXIMUM FLOW FOR DESIGN
*C
*****
```

I  
I  
I

```
*****
* DO 105 I=1, NN
*****
```

I  
I  
I

```
*****
* OLDDIA(I)=DIA(I)
* KK=ALIMIT(I)/DELTA
* MAX(I,1)=0.0
* FLOW(I,1)=0.0
* IF(KK.GE.MAXTI) KK=MAXTI
*****
```

I

A.....\*  
A  
A  
A  
A  
A  
A  
A  
A  
A  
A  
A

[illegible]



A		I	B	F	
A		I	B	F	
A		I	B	F	
A		I	B	F	
A		*****	B	F	
A		* 110 NANING(I)=ABS(NNLOW(I)-NNHIGH(I))/2.0+NNLOW(I)	B	F	
A		* PSLOP=PSLOPE(I)+0.01	B	F	
A		* DIAM(I)=(2.16*NANING(I)* AXFLOW(I))/(PSLOP**0.5)**0.375	B	F	
A		* DIA(I)=DIAM(I)*12.0	B	F	
A		* MM(I)=DIA(I)*1.0	B	F	
A		* MM=MM(I)	B	F	
A		*****	B	F	
A		I	B	F	
A		I	B	F	
A		*****	B	F	
A	B.....*	DO 150 I=1,MM	B	F	
A	B	*****	B	F	
A	B	I	B	F	
A	B	I	B	F	
A	B	*****	B	F	
A	B	* DTOD(I)=FLOAT(I)/MM(I)	B	F	
A	B	*****	B	F	
A	B	I	B	F	
A	B	I	B	F	
A	B	*****	B	F	
A	B	* IF(DTOD(I).GE.0.0.AND.DTOD(I).LE.0.2) GO TO 115	B.....*	F.....0	
A	B	*****	B	F	J
A	B	I	B	F	J
A	B	I	B	F	J
A	B	*****	B	F	J
A	B	* IF(DTOD(I).GT.0.2.AND.DTOD(I).LE.0.5) GO TO 120	B.....*	F.....J.....0	
A	B	*****	B	F	J
A	B	I	B	F	J
A	B	I	B	F	J
A	B	*****	B	F	J
A	B	* IF(DTOD(I).GT.0.5.AND.DTOD(I).LE.0.8) GO TO 125	B.....*	F.....J	N
A	B	*****	B	F	J
A	B	I	B	F	J
A	B	I	B	F	J
A	B	*****	B	F	J
A	B	* IF(DTOD(I).GT.0.8.AND.DTOD(I).LE.1.0) GO TO 130	B.....*	F.....J	N
A	B	*****	B	F	J
A	B	I	B	F	J
A	B		B	F	H
A	B		B	F	H
			B	F	H
			B	F	H

A B	I	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	0<.....	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	* 115 SS(I)= (DTOD(I)-0.0)/0.05	B	D	F	H	J	N
A B	* PN(I)=1.0+SS(I)*0.18+SS(I)*(SS(I)-1.0)*(-0.065)+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*0.01833+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*(SS(I)-3.0)*(-0.0041667)	B	D	F	H	J	N
A B	* MANING(I)=PN(I)*NNLOW(I)	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	* GO TO 135	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	0<.....	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	* 120 SS(I)= (DTOD(I)-0.5)/0.05	B	D	F	H	J	N
A B	* PN(I)=1.24+SS(I)*(-0.015)+SS(I)*(SS(I)+1.0)*(SS(I)+2.0)	B	D	F	H	J	N
A B	* +0.00167	B	D	F	H	J	N
A B	* +SS(I)*(SS(I)+1.0)*(SS(I)+2.0)*(SS(I)+3.0)*0.0008333+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)+1.0)*(SS(I)+2.0)*(SS(I)+3.0)*(SS(I)+4.0)*	B	D	F	H	J	N
A B	* 0.00025+SS(I)*(SS(I)+1.0)*(SS(I)+2.0)*(SS(I)+3.0)*	B	D	F	H	J	N
A B	* (SS(I)+4.0)*(SS(I)+5.0)*0.000041667	B	D	F	H	J	N
A B	* MANING(I)=PN(I)*NNLOW(I)	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	* GO TO 135	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	0<.....	B	D	F	H	J	N
A B	I	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	* 125 SS(I)= (DTOD(I)-0.5)/0.05	B	D	F	H	J	N
A B	* PN(I)=1.24+SS(I)*(-0.01)+SS(I)*(SS(I)-1.0)*(-0.005)+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*0.0025+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*(SS(I)-3.0)*(-0.0008333)+	B	D	F	H	J	N
A B	* SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*(SS(I)-3.0)*(SS(I)-4.0)*	B	D	F	H	J	N
A B	* 0.000125+SS(I)*(SS(I)-1.0)*(SS(I)-2.0)*(SS(I)-3.0)*	B	D	F	H	J	N
A B	* (SS(I)-4.0)*(SS(I)-5.0)*0.000013889	B	D	F	H	J	N
A B	* MANING(I)=PN(I)*NNLOW(I)	B	D	F	H	J	N
A B	*****	B	D	F	H	J	N
A B	I	B	D	F	H	J	N

[illegible]

[illegible]

```

A B      D<.....0
A B      I
A B      .....
A B      * 185 SS(II)=(DTOD(III)-0.5)/0.1
A B      * HYDRAD(III)=(0.240*DIAH(III)*1.05+SS(III)*0.075+SS(III)*(SS(III)-1.0)*
A B      * (-0.015)+SS(III)*SS(III)-1.0)*SS(III)-2.0)*(-0.0016667))*
A B      * C SS(III)*SS(III)-1.0)*SS(III)-2.0)*SS(III)-3.0)*(-0.00020833)*
A B      * C )
A B      .....
A B      I
A B      D<.....F.....J.....N
A B      I
A B      .....F.....J.....N
A B      * 190 CALL VELIMM,LENGTH,PSLOPE,TIMSEW,I,II,NN,GOOD,EPSILN,OLDDIA,DIA,
A B      * & ZEDA,ANGLE,WETPER,AREAFI,MANING,LONG,SS,PN,OTOD,Y,X,DEEP,
A B      * & HYDRAD,VELOCITY,TIMSER,TOTTIS,AVETIM,MAXDIA)
A B      .....F.....J.....N
A B      I
A B      I
A B      I
A B      .....F.....J.....N
A B      * 195 CONTINUE
A B      .....F.....J.....N
A B      I
A B      I
A B      I
A B      .....F.....J.....N
A B      * GO TO 220
A B      .....F.....J.....V
A B      .....F.....J.....N
A B      D<.....0
A B      I
A B      .....J.....N
A B      * 200 NANING(I)=ABS(NNLOW(I)-NNHIGH(I))/2.0*NNLOW(I)
A B      * PSLOP=PSLOPE(I)*0.01
A B      * DIAH(I)=(0.532685*NANING(I)*AXFLOW(I))/(PSLOP*0.5))**0.375
A B      * DIA(I)=DIAH(I)**12.0
A B      * MM(I)=DIA(I)**1.0
A B      * DIAM(I)=MM(I)/12.0
A B      * LONG(I)=2.0*DIAH(I)
A B      * MMM=MM(I)
A B      .....J.....N
A B      I
A B      I
A B      I
A B      .....J.....N
A B      * DO 215 II=1,MMM
A B      .....J.....N
A B      I

```

```

*****
*      DTOD(11)=FLOAT(11)/MH(1)
*****
*****
*****
*      IF(DTOD(11).GE.0.0.AND.DTOD(11).LE.0.5) GO TO 205
*****
*****
*****
*      IF(DTOD(11).GT.0.5.AND.DTOD(11).LE.0.9) MANING(11)=0.70*NNHIGH(1)
*      IF(DTOD(11).GT.0.9.AND.DTOD(11).LE.1.0) MANING(11)=NNHIGH(1)
*****
*****
*****
*      GO TO 210
*****
*****
*****
*      205 SS(11)=DTOD(11)/0.1
*      MANING(11)=NNHIGH(11)*(1.0+SS(11))*(-0.15)+SS(11)*(SS(11)-1.0)*(0.0
*      4)+(SS(11)*(SS(11)-1.0)*(SS(11)-2.0)*(-0.00833)+SS(11)*
*      1)SS(11)-1.0)*(SS(11)-2.0)*(SS(11)-3.0)*(0.0014583)+SS(1
*      1)*(SS(11)-1.0)*(SS(11)-2.0)*(SS(11)-3.0)*(SS(11)-4.0)*
*      (-0.002083))
*****
*****
*****
*      210 DEEP(11)=(11+1.0)/12.0
*      AREAFL(11)=DEEP(11)*(LONG(11))
*      HYDRAD(11)=AREAFL(11)/(LONG(11)+2.0*DEEP(11))
*      CALL VEL(MMH,LENGTH,PSLOPE,TIMSEW,1,11,NN,GOOD,EPSILN,OLDDIA,DIA,
*      6,ZEDA,ANGLE,BETPER,AREAF,MANING,LGNG,SS,PN,DTOD,Y,X,DEEP,
*      6,HYDRAD,VELOCITY,TIMSER,TOTTIS,AVETIM,MAXDIA)
*****

```

```
A B J  
A B J  
A B J  
A B J  
A ..... * 215 CONTINUE *
```

A  
A  
A  
A  
A  
A  
A

..... \* 220 CONTINUE \*

A  
A  
A  
A  
A  
A  
A

..... \* 225 CONTINUE \*

A  
A  
A  
A  
A  
A  
A

..... \* RETURN \*

A  
A  
A  
A  
A  
A  
A

..... \* END \*

(ENTRANCE)

```
*****
* SUBROUTINE VEL(MMM,LENGTH,PSLOPE,TIMSEW,I,II,NN,GOOD,EPSILN,
* C OLODIA,DIA,ZEDA,ANGLE,WETPER,AREAF,MANING,LUNG,SS,PN,DTOD,Y,X,
* C DEEP,HYDRAD,VFLOCY,TIMSER,TOTTIS,AVETIN,MAXDIA)
* REAL S,NNLOW,NNHIGH,LTAND,MANK,LENGTH
* REAL INTEN,MAN,NANING,MAX,MANING,LONG
* INTEGER ALIMIT,AINTER
* DIMENSION ZEDA(MAXDIA),ANGLE(MAXDIA),WETPER(MAXDIA),
* C AREFL(MAXDIA),MANING(MAXDIA),LONG(NN),SS(MAXDIA),
* C PN(MAXDIA),DTOD(MAXDIA),Y(MAXDIA),X(MAXDIA),DEEP(MAXDIA),
* C HYDRAD(MAXDIA),VELOCITY(MAXDIA),TIMSER(MAXDIA),TOTTIS(NN),
* C AVET(MNN)
* DIMENSION LENGTH(NN), PSLOPE(NN),TIMSEW(NN),GOOD(NN),EPSILN(NN),
* C OLODIA(NN),DIA(NN)
* PSLOP=PSLOPE(I)*0.01
* VELOC(I)=(1.49/MANING(I))*[HYDRAD(I)*0.6667]*(PSLOP*0.5)
* TIMSER(I)=LENGTH(I)/VELOC(I)*60.0
*****
I
I
*****
* IF(II.EQ.1) GO TO 100
*****
I
I
*****
* TOTTIS(I)=TIMSER(I)+TOTTIS(I)
*****
I
I
*****
* GO TO 105
*****
DC.....0
I
*****
* 100 TOTTIS(I)=TIMSER(I)
*****
DC.....0
I
*****
* 105 IF(II.NE.MMM) GO TO 110
*****
DC.....0
I
```



```

      I
      I
      I
*****
*      AVETIM(I)=TOTYIS(I)/(MMH)
*      EPSILN(I)=ABS(OLDOIA(I)-D(A(I))/D(A(I))
*      TIMSEW(I)=AVETIM(I)
*      IF(EPSILN(I).LE.0.15) GOOD(I)=1.0
*****
      I
      I
*****
*      IF(EPSILN(I).LE.0.15) GO TO 110
*****
      I
      I
*****
*      GOOD(I)=0.0
*****
      I
      0<.....0
      I
*****
*      110 RETURN
*****

*****
*      END
*****

```